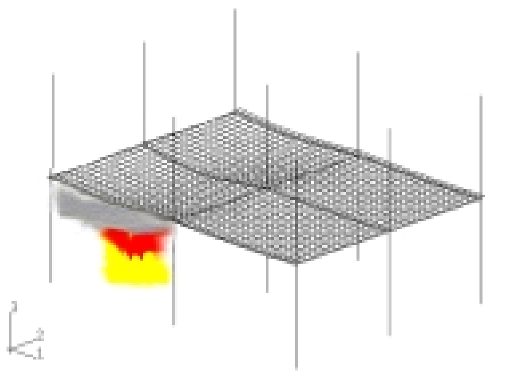
Helsinki University of Technology Laboratory of Steel Structures Publications 18

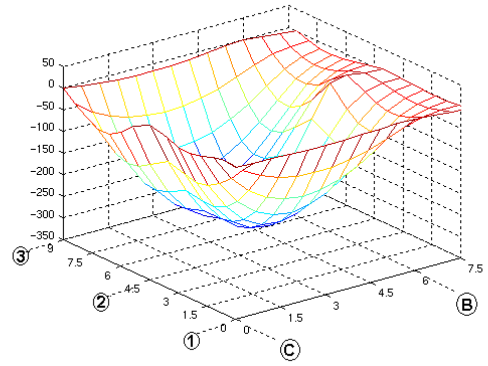
Teknillisen korkeakoulun teräsrakennetekniikan laboratorion julkaisuja 18

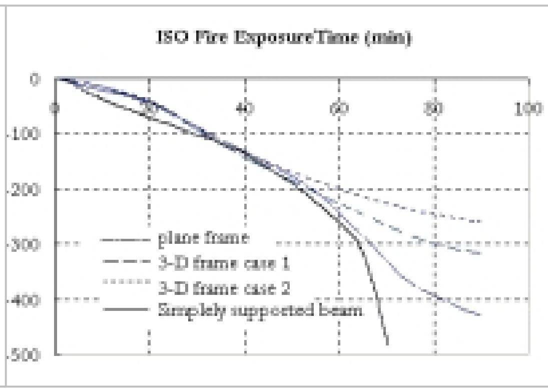
Espoo 2000 TKK-TER-18

FIRE SAFETY DESIGN OF COMPOSITE SLIM FLOOR STRUCTURES

Zhongcheng Ma







TEKNILLINEN KORKEAKOULU
TEKNISKA HÖGSKOLAN
HELSINKI UNIVERSITY OF TECHNOLOGY
TECHNISCHE UNIVERSITÄT HELSINKI
UNIVERSITE DE TECHNOLOGIE D'HELSINKI

Helsinki University of Technology Laboratory of Steel Structures Publications 18

Teknillisen korkeakoulun teräsrakennetekniikan laboratorion julkaisuja 18

Espoo 2000 TKK-TER-18

FIRE SAFETY DESIGN OF COMPOSITE SLIM FLOOR STRUCTURES

Zhongcheng Ma

Dissertation for the degree of Doctor of Science in Technology to be presented with due permission for public examination and debate in Auditorium R1 at Helsinki University of Technology (Espoo, Finland) on the 7th of December, 2000, at 12 o'clock noon.

Helsinki University of Technology

Department of Civil and Environmental Engineering

Laboratory of Steel Structures

Teknillinen korkeakoulu

Rakennus- ja ympäristötekniikan osasto

Teräsrakennetekniikan laboratorio

Distribution:
Helsinki University of Technology
Laboratory of Steel Structures
P.O. Box 2100
FIN-02015 HUT
Tel. +358-9-451 3701
Fax. +358-9-451 3826
E-mail: sinikka.rahikainen@hut.fi

ISBN 951-22-5222-8
ISSN 1456-4327

Otamedia Oy 2000

Preface

This study has been carried out in the Laboratory of Steel Structures at Helsinki University of Technology. I wish to express my gratitude to my superior, Professor Pentti Mäkeläinen, who has enabled my study and provided the financial support for the research work.

My cordial thanks to assistant Lic. Sc. (Tech.) J. Kesti, who arranged the facilities for the research work. Sincere thanks also to my colleagues Misters M. Malaska, J. Outinen and O. Kaitila.

Special thanks are due to Mrs Sinikka Rahikainen, our secetary with a very kind heart.

Thanks to Mr. N. Kimberley for the language checking of the manuscript.

Finally I would like to thank my wife, Wei Lu, for the 'joint' research life. Uncountable misses to my Mom and Dad, who have almost lost their son since the start of my overseas studying life. Many thanks to my parents-in-law, who have encouraged me to study abroad, to seek being a man with open insight.

This work has been financially supported by the Centre for International Mobility (CIMO), the National Technology Agency of Finland (TEKES), The Finnish Constructional Steelwork Association Ltd., and the Finnish steel company - Rautaruukki Oyj, which I wish to acknowledge with gratitude.

Espoo, October 2000

Zhongcheng Ma

Abstract

A structural fire safety design method for composite steel-concrete slim floor structures was developed, including the characterization of fully-developed compartment fire curves and the equivalent fire exposure, temperature analysis of the composite structures, and the structural response analysis of composite slim floor structures subjected to fire. Special interest was given to the structural responses of the slim floor beam both as an isolated member and a part of the frame structure. The mechanical interaction behind the global deformation response of the frame was emphasized.

Although the analyzed objects concentrated on composite slim floor structures, the according fire safety design method and the explored structural response mechanism in fire are equally applicable to other types of building structures.

Keywords: Fire safety design, parametric fire curves, equivalent fire exposure, fire resistance, slim floor, composite frame, numerical modelling

Contents

List of Publications

This dissertation consists of an overview, lastest research results on the parametric fire temperature – time model and its equivalent fire exposure, the structural behaviour of the composite slim floor frame in fire, and the following publications:.

A. Ma, Z. and Mäkeläinen, P. Parametric Temperature-time Curves of Medium Compartment Fires for Structural Design. Fire Safety Journal, vol. 34, no. 4, pp.361-375, June 2000.

B. Mäkeläinen, P. and Ma, Z. Fire Resistance of Composite Slim Floor Beams. Journal of Constructional Steel Research, vol. 54, no. 3, pp.345-363, June 2000.

C. Ma, Z. and Mäkeläinen, P. Behaviour of Composite Slim Floor Structures in Fire. Journal of Structural Engineering, ASCE, vol. 126, no.7, pp.830-837, July 2000.

D. Ma, Z. and Mäkeläinen, P. Temperature Analysis of Steel-Concrete Composite Slim Floor Structures Exposed to Fire. The 6th. ASCCS International Conference on Steel-Concrete Composite Structures - the past and the coming century, March 22-24 2000, Los Angeles, volume I, pp.263-270.

Author's Contribution

Some of the research results on the parametric fire temperature – time model and its equivalent fire exposure, and the structural behaviour of the composite slim floor frame in fire which are presented in the context in more details, are published at the first time by the author. The other research related to the topic by the author can be found in publications 109-117 and 187-192. The research work and preparation of the manuscripts in all papers A, B, C and D were carried out alone by Zhongcheng Ma. The other author, Professor Pentti Mäkeläinen, has acted only as the supervisor of the research project.

Chapter 1

Introduction

1.1 General

Fire safety design is an important aspect of building design. A properly designed building system greatly reduces the hazards to life and limits property loss. The research on fire safety design started as long ago as almost 80 years (Inberg 1928). Today the concept of fire safety design has been improved significantly and become more rational.

Current fire safety concepts are optimally integrated packages of passive structural, active technical and active organizational fire precaution measures, which allow well-defined objectives agreed by the owner, the fire authority and the designer to be fulfilled.

Passive structural measures include fire-resistant load-bearing structures, the fire protection of the main load-bearing elements and fire compartimentation by floors and walls. Active technical measures include automatic fire detection, automatic fire alarm transmission devices, fire-fighting installations and automatic fire suppression devices such as sprinklers, etc. Active organizational measures include water supply, public and on-site fire fighting services, installations for means of escape, training, emergency plans and so on.

The fire safety objectives in the present European Fire Codes are explicitly based on the life safety objective [35]. However, protecting properties from loss due to the careless behaviour of a neighbour is another important aim. According to the Construction Product Directive (CPD)[34] in the clause concerning 'Safety in case of fire', the following essential requirements have to be fulfilled in the event of an outbreak of fire

- the load-bearing capacity of the construction can be assumed for a specific period of time
- the generation and spread of fire and smoke within the works are limited
- the spread of fire to neighbouring construction works is limited
- occupants can leave the works or be rescued by other means
- the safety of rescue teams is taken into consideration

The central objective of fire safety in the current Fire Codes is to confine the fire within the compartment in which it started. They consist of a collection of requirements, only or mostly related to the structural fire resistance of load-bearing elements and to walls and slabs necessary to guarantee the compartmentation.

It should be noted that the objectives of fire safety are a historical concept, in which the content can change with the development of fire science. Besides, the additional objectives (e.g. reparability) can also be implemented if the client or authorities require a particular building or project.

1.2 Structural Fire Safety Design Methods

Structural fire safety design contains two important aspects, including the characterization of fully developed compartment fires and verification of the ability of structures to resist and contain fires. Currently, the design methods may be classified into two classes:

- Methods related to fire resistance only
- Methods related to global fire safety

The first category of methods concerns the verification methods of fire resistance. The structural Eurocodes are for the time being strictly limited to this category. The second category is based on the fire risk assessment technology, which is being developed for particular buildings, important structures or individual projects, in which the fire safety objectives are required by the owner, authorities and designer.

Generally, the first category of methods is simple to design, but the safety margin for individual building is uncertain. The second category of methods is complex to design, but the safety margin is known and it is cost-effective as well.

1.2.1 *Methods Related to Fire Resistance only*

The methods related to fire resistance only are combined by fire models and structural models. The fire models define the evolution of air temperature, the convective and radiative boundary conditions, and the spreading of fire in a fire-affected room if possible. The structural models define elements or parts of the structures, thus allowing the prediction of the temperature increase in the structure or in elements ensuring compartmentation, of the collapse temperature or the collapse time for a given load.

Zone modelling and field modelling (Computational Fluid Dynamics) are two prevalent modelling techniques used to simulate the fire scenario in a building compartment. Several commercial software packages such as CFAST by NIST, HAZARD by Harvard, JASMINE by BRE, and SOFIE by a consortium of European fire research laboratories, have been developed for the modelling of fire in a building compartment. However, in view of the structural fire safety design, only the post-flash-over fire has a significant influence on the structural stability and integrity. These modelling techniques offer a high degree of accuracy in calculating the temperature evolution of a compartment fire but are extremely sophisticated. They are not very convenient tools for structural fire safety design.

An approximate method to describe the temperature evolution of a fully developed compartment fire has been considered desirable. The current Eurocode 1 gives a group of nominal fire curves including a standard fire curve (ISO 834), an external fire curve and a hydrocarbon fire curve. These curves are only time-dependent. As an alternative, Eurocode 1 also gives a set of parametric fire curves for relatively small fire compartments. These curves depend on the fire load, opening factor, thermal properties of boundaries and the geometry of the fire compartment.

In the current codes, three kinds of structural models are normally used

- Simple calculation on elements like column and beam, with simple support conditions;
- Taking account of a sub-system with appropriate boundary conditions;
- Analyzing the global structural response to fire.

Any one of the three models can be taken for the thermal and structural calculation of fire resistance. However, in the first model, the buildings are treated as a series of individual members, and the continuities and interaction between these members are assumed to be negligible. Throughout the 1990s, following the investigation of the fire event in Broadgate (1991, UK), fire tests in William Street (1992, Australia), and full-scale fire tests on an 8-storey composite steel-framed building in Cardington (1995, 1996, UK), it was found that the structural member in the frame exhibited significantly better behaviour in fire than that in the standard fire-resistance test. The first model is very conservative by disregarding the interaction between members. The fire event and tests also highlighted that the first model, although conservative, did not address the true behaviour of the building structure in fire, since the building was not acting as a series of individual members. However, this model is simple to design.

The third model is an advanced analysis method by considering the building structure as a whole. The restraints by the surrounding parts and interaction between the structural members can be taken into account. However, the analysis is complicated and time-consuming. For the time being this model is mostly used for research purpose or for the individual project. The second model is in between the other two.

1.2.2 *Methods Related to Global Fire Safety*

As far as this category is concerned, the probabilistic and statistical methods are used to determine the probability of occurrence and the severity of fire in a building. The active fire measures such as heat/smoke detection, fire alarm, sprinklers, automatic closure of doors leading to compartmentation and fire-fighting services, in conjunction with the risk of fire occurring in the building - such as in a painting workshop or a swimming pool -, and the mobility of people in the building, will affect the severity of the fire design (including intensity and duration).

There are several different fire risk assessment methods developed respectively by Switzerland, Australia, the USA, Denmark, Canada and the ECCS. The ECCS method is one based on the logic tree analysis, which concerns with the probabilistic evaluation of the occurrence and spreading of a fire scenario in a building. In this method, the event 'fire' is divided into sub-events, which are easier to analyze. The occurrence probability of a fully developed fire can be evaluated, and thus the probabilistic fire curve can be defined.

The structural response to the possible fire can be analyzed by the advanced structural analysis techniques described above. The stability of the structures and integrity of the compartmention can be checked according to the criteria of the codes.

1.3 Development of Composite Slim Floor Structures

In recent years, increasing interest has been shown throughout Europe in developing and designing shallow floor systems in steel-framed buildings. In the

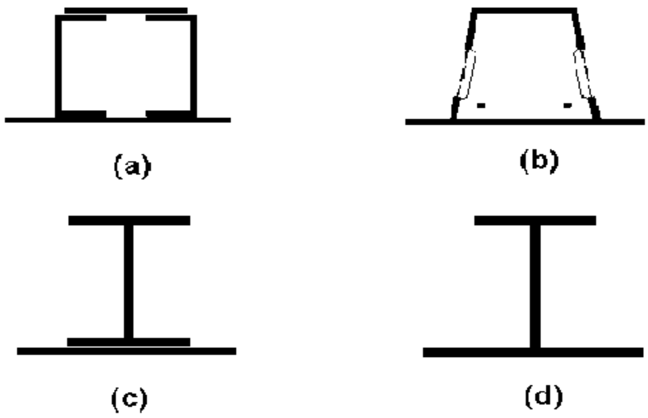


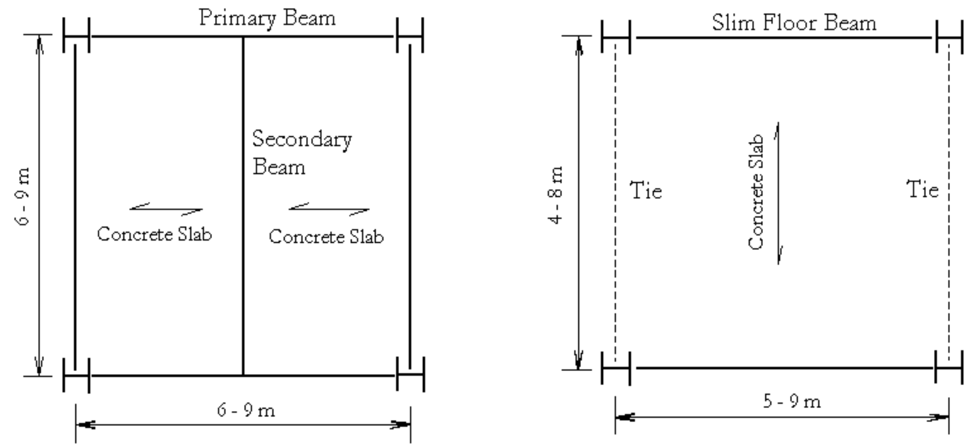
Fig. 1.1 Different Types of Slim Floor Beams: (a) Thor-Beam; (b) Delta-Beam; (c) Slimflor Beam; (d) Asymmetric Slim Floor Beam.

shallow floor system, the steel beam is contained within the depth of the pre-cast concrete floor or composite slab with profiled steel decks. This form of construction achieves a minimum depth of building and the flat floor is beneficial because the building services can be run in any direction. The shallow floor can be designed using various forms of steel beams comprising either rolled or welded sections, which are called 'slim floor beams'.

The key features of the composite slim floor construction are the steel beam and the type of steel deck. One of the original slim floor concepts developed in Scandinavia was the 'Thor-beam' (Fig. 1.1a), which consists of two channel sections welded to a flat plate. Additional angles are welded to the top flanges to provide the shear connection. Another concept is the 'Delta-beam' (Fig. 1.1b). The slim floor system consisting of the 'Delta-beam' and prefabricated concrete slab is very popular in Finland. In 1991, British Steel and The Steel Construction Institute (SCI) developed a Slimflor® beam (Fig. 1.1c), which consists of a universal column section welded to a steel plate. Recently, interest has been concentrated on the asymmetric hot-rolled steel beam (Fig. 1.1d) in the UK, and on the asymmetric welded steel beam in Finland.

The slim floor beam offers good fire resistance because most of the steel beam is encased within the depth of the concrete slab. Several fire tests performed by SCI (Steel Construction Institute) in the UK [95,127,131] have shown that the composite slim floor beam can achieve 60 minutes' fire resistance without any additional fire protective measures if the load ratio is less than 0.5. To achieve 90 minutes' fire resistance, only a very thin protection underneath the lower flange is needed [B].

Generally, the conventional steel-concrete composite beam is well established for longer spans (> 9 m), but the slim floor beam creates more opportunities for steel in spans of 5-9m. It also achieves a slab depth of 300 mm or so, which is much less than that in conventional steel construction. This issue increases the competitiveness of composite slim floor construction with the concrete flat slab system. On the other hand, compared with the conventional composite frame system that has a primary-secondary-beam system, the new slim floor frame has a rather precise structural form



(a) **(b)**

Fig. 1.2 Composite Frame Systems: (a) Traditional Primary - Secondary Beams System; (b) Slim Floor System.

(Fig. 1.2). In slim floor construction, the slab is supported directly by the primary beam, and forms a part of the composite beam to work together with steel beam. Between the rows of the single frame, the tie members are employed to link them together and maintain the out-of-plane stability of the frame.

1.4 Aims of the Present Study

The aim of the present research project is to investigate fire resistance of the composite asymmetric slim floor beam both as an isolated member and as a part of the frame using numerical analysis methods. As an isolated element, the research includes the fire resistance of the new beam and the effects of enhanced measures such as additional reinforcements and fire paints underneath the bottom flange. In the frame analysis, the research focuses on the realistic behaviour of the slim floor frame as a whole in fire. The effects of frame continuity and its quantification, mechanical interaction between the structural members and the role of floor system on the stability of the whole building in fire have been investigated.

Another aim of the study is to develop a new parametric temperature-time curve for structural design, or improve the present parametric curve in Eurocode, to consider the situation in the fuel-controlled region and so on. The according method of equivalent fire exposure will also be developed.

Chapter 2

Parametric Temperature-Time Curves for Structural Design

2.1 Behaviour of Fires in Buildings

2.1.1 *Introduction*

The temperature-time curves in a fire compartment designate the characteristics and intensity of fire developing process. A typical temperature development in the compartment is shown in Fig. 2.1, which includes three phases: fire growth, full development and decay.

In the fire growth phase, the room temperature is low and the fire is local in the compartment. The period is important for evacuation and fire fighting. Usually, it is not of significant influences on the structures. After flashover, the fire enters into the fully developed phase, in which the temperature of the compartment increases rapidly and the overall compartment is engulfed in fire. The highest temperature, highest rate of heating and largest flame occur during this phase, which gives rise to the most structural damage and much of the fire spread in buildings. In the decaying period, the temperature decreases gradually. It is worth pointing out that this period is also important to the structural fire engineering because for the R.C. structures, composite structures, insulated steel structures and unprotected steel structures of low section factor, the internal temperature of cross-section will still increase significantly even though it is the decaying period.

The efforts to quantify fire characteristics can be seen as far back as 1928 in Inberge's study [77]. From efforts spanning almost 80 years, considerable knowledge about the fire has been obtained. The numerical modelling of fire scenarios for medium and large compartments has also produced many successful experiences. A one-zone model, multi-zone model and computational field model (CFD) have been developed and commercial software is available. However, the overview in this section will only focus on those relevant to the characterization of fire temperature evolution in a compartment.

The factors that have a significant influence on compartment fires are fire load and type, ventilation opening, and the geometry of fire compartment. The thermal properties of the compartment boundary also have some effect on the fire, and the variation on the maximum temperature is of the order of 10% [76, 94].

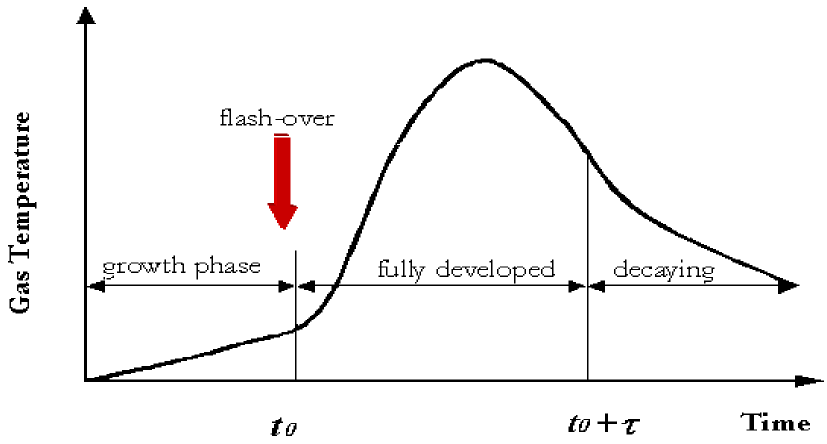


Fig. 2.1 Typical Temperature Development in a Compartment.

2.1.2 *Regimes of Burning*

The first systematic study of the behaviour of the fully developed compartment fire was carried out in Japan in the 1940s by Kawagoe et al. [81, 82]. In the study, the burning rate of wood cribs was found to depend strongly on the size and shape of the ventilation opening, which was called 'ventilation-controlled fires'. However, Thomas et al. [170] found that, if the ventilation opening was enlarged, a condition would be reached beyond which the burning rate would be independent of the size of the opening and would be determined instead by the surface and burning characteristics of the fuel, which was called 'fuel-controlled fires'.

Harmathy [73] analyzed data from a large number of wood crib fires in a compartment and plotted the results as R/A_{fuel} versus $\rho_a g^{1/2} A_w h^{1/2}/A_{fuel}$. This plot shows a clear distinction between the 'ventilation-controlled regime' and a 'fuel-controlled regime' in which the burning rate R is independent of the ventilation factor (Fig. 2.2). Harmathy recommended the following conditional equation

$$\frac{\rho_a g^{1/2} A_w \sqrt{h}}{A_{fuel}} = 0.235 \sim 0.290 \tag{2.1}$$

where ρ_a is the density of air (kg/m^3); g is the acceleration due to gravity (m/s^2); A_w and h are the area (m^2) and height (m) of the opening respectively; A_{fuel} is the surface area of the fuel (m^2).

2.1.3 *Rate of Burning*

The rate of heat release, which is the product of the rate of burning and the calorific value of fuel, is one of the most important characteristics of a fire. In some one-zone and multi-zone modelling, the rate of heat release of the fuel has to be input as the basic given variable. It represents the released heat by the fuel per unit of time

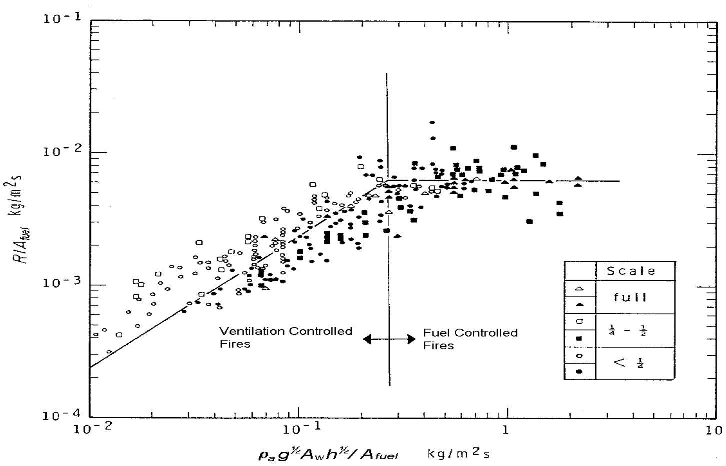


Fig. 2.2 Identification of the Transition Region between Ventilation-controlled and Fuel-controlled Fires for Wood Cribs [73].

In fact, in most experiments the rate of weight loss of the fuel was measured, rather than heat release; and although the rate varies during the course of a fire, it was found that during the fully developed period it had a steady value R. Generally, R was defined as the average burning rate of fuel beginning when the fuel mass had dropped to 80% of its original value and ending at 30%. This period is also known as a 'primary burning'.

The earlier experimental results in Japan in the 1940s and later in the former USSR, the USA and the UK showed that R was mainly controlled by the rate of airflow that enters the compartment. This led to the well-known equation $R=0.1A_w h^{1/2}$ (kg/s). Later, this equation was verified by the theoretical derivation of heat balance in a fire compartment, under the specified restraint conditions [3]. Harmathy analyzed the experimental results for wood cribs and found that the rate of burning was well correlated to the surface area of fuel for fuel- controlled fires (Fig. 2.2). Further analysis on CIB fire tests by Law et al. [91,92] showed that, for ventilation-controlled fires, other features should be taken into account, notably the area of the enclosing surfaces of the compartment and the depth D of the compartment in relation to the width W of the wall containing the window (Fig. 2.3).

In general, the rate of burning of wood-based compartment fires can be expressed as:

For ventilation-controlled fires,

$$R = 10.8 \cdot (1 - exp(-0.036 \cdot \eta)) \cdot A_w \sqrt{h} \cdot \sqrt{W / D} \qquad (2.2)$$

and for fuel-controlled fires as,

$$R = 0.372 \cdot A_{fuel} \qquad (2.3)$$

where R is in kg/s.

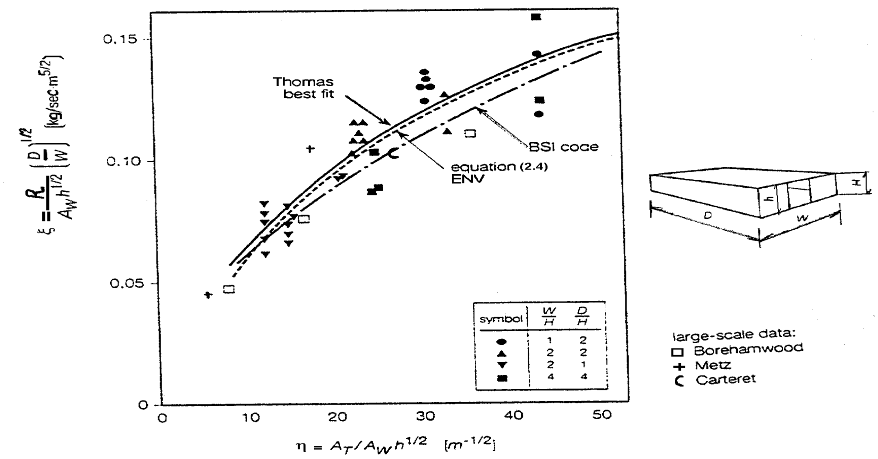


Fig. 2.3 Rate of Burning for Ventilation Controlled fires [92, 168].

2.1.4 *Maximum Average Compartment Temperature and Fire Duration*

The air temperature and fire duration are two important factors in characterizing fire severity. The maximum average air temperature represents the intensity of a compartment fire. Ma and Mäkeläinen [A, 114] analyzed the data from fully developed compartment fire tests for wood-cribs fires, and found that for fuel-controlled fires, the fire intensity had a good correlation with the ratio of opening factor $A_w h^{1/2} / A_t$ (where A_t is the total surface area of compartment boundary, m^2) and the critical opening factor (determined by Eq. 2.1). The following equations were given

For ventilation-controlled fires,

$$T_{g\max} = 1240 - 11 \cdot \eta \tag{2.4}$$

and for fuel-controlled fires,

$$\frac{T_{g\max}}{T_{gmcr}} = \sqrt{\frac{\eta}{\eta_{cr}}} \tag{2.5}$$

where T_{gmax} is the maximum average air temperature in a compartment (°C); $\eta = A_t / A_w h^{1/2}$, the inverse of opening factor ($m^{-1/2}$); η_{cr} is the value of η in the critical region, which can be obtained from Eq. 2.1. T_{gmcr} is the maximum temperature in the critical region (°C), which can be obtained by substituting η_{cr} into Eq. 2.4.

Thomas et al. [172] gave an expression on the maximum air temperature in a compartment fire, after the analysis of data from CIB small-scale fire tests. However, the critical regime of two kinds of fires was ill defined. In the expression, the critical regime was dependent only on a constant of opening factor (η=12.5 $m^{-1/2}$), which was not correlated to the fuel.

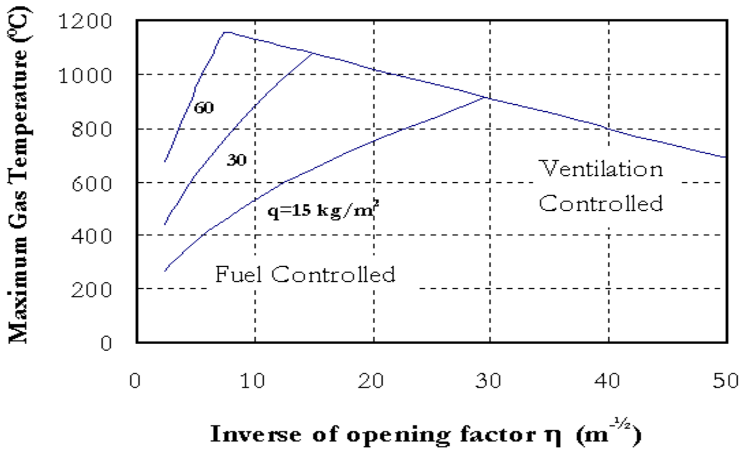


Fig. 2.4 Maximum Average Gas Temperature in a Compartment [A]. (The other parameters: the ratio of floor area to the total surface area enclosing the compartment is 3/14, and the surface area of unit weight of fuel (equavilent woods) is 0.15 m²/kg).

Although fire duration is an extensively used phrase, it is not clearly defined up to today. Intuitively, the fire duration could be defined as

$$\tau = G / R \tag{2.6}$$

where G is the total weight of fuel (equivalent woods, kg).

2.2 Parametric Fire Models for Structural Design

2.2.1 *Nominal Fire Curves*

The nominal fire curves are a set of temperature-time relationships, in which no physical parameters are taken into account. The main purpose of the prescription of the nominal fire curves was to make the fire resistance tests reproducible. The ability of fire resistance of building elements can be evaluated under the same heating curve. The current Eurocode gives three nominal fire curves: ISO standard fire curve, external fire curve and hydrocarbon fire curve. The first two are for cellulosic fire and the third is for hydrocarbon based fire.

2.2.2 *Parametric Fire Curves based on Heat Balance*

Several groups (for instance, Kawagoe's, Pettersson's and Babrauskas's) have attempted to predict the temperature-time history in a compartment fire, for the purpose of structural fire resistance design, based on the heat and mass balances of the compartment. The earlier curves by Kawagoe et al. [82] were derived only from the heat balance equations. The constant rate of burning was used and it was assumed that there was an unlimited supply of fuel. Pettersson et al. [142] complemented the mass equation, and used the realistic rate of burning derived from the measured data. Nevertheless, the rate of burning in his analysis was still only ventilation controlled. Using this model, a set of temperature-time curves was developed for the convenience of the designer, which were in the parameters of fire load density, opening factor and thermal properties of compartment boundary.

Franssen etal [64] attempted to take account of the case for fuel-controlled fires as well. A one-zone model was used and the combustion model could define the burning regimes. The computer program, *OZONE,* was coded and a set of temperature-time curves in the parameters of fire load density and opening factor were calculated. These curves were stored on the PC disk in the form of differentiated data.

Another attempt made by Babrauskas et al. [5] gave an approximate mathematical expression by fitting the formulae with the calculated results using the *COMPF2* computer code. This method requires the rate of burning as input, which is still difficult to determine nowadays.

2.2.3 *Parametric Fire Exposure in Eurocode*

With the thinking that the methods (Kawagoe's, Pettersson's and Babrauskas's) were sophisticated and that the assumptions and uncertainties associated with the models were easy to make them premature, Lie (1974) suggested that it is only necessary find a fire temperature-time curve 'whose effect, with reasonable probability, will not be exceeded during the use of the building', if the objective is to develop a method of calculating fire resistance. He developed an expression based on a series of temperature-time curves computed by Kawagoe et al. It certainly inherited the assumption of unlimited fuel supply made in Kawagoe's computation, before the fire duration was reached.

Having a similar idea to Lie's, Wickström developed an expression by 'applying the standard fire curve for expressing natural fires for design purposes'. The current Annex B of Eurocode 1 [55] is based on his work. Compared with Lie's expression, which is only dependent on the opening factor, the method in Eurocode takes account of all the four significant fire parameters. However, it is very poor for fuel-controlled fires [114].

The original temperature-time equation in Annex B of Eurocode 1 is given by

$$T_g = 1325 \cdot \left(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*}\right) \tag{2.7}$$

where $t^* = \Gamma \cdot t$, t is time in hours; $\Gamma = (O/b)^2 / (0.04/1160)^2$, O is the opening factor ($m^{½}$), b is the thermal properties of enclosing boundary ($= \sqrt{\rho\lambda c}$, $Jm^2s^{½}K$). The time corresponding to maximum temperature is $t_d = 0.13 \times 10^{-3} \cdot q_t / O$, in which $q_t = q \cdot A_f / A_t$. q is the fire load density per unit floor area (MJ/m^2). After the maximum temperature is reached, the temperature declines linearly and the slope depends on the value of t_d^* ($= \Gamma \cdot t_d$).

2.2.4 *Parametric Fire Curves based on Experimental Approximation*

An attempt has been made by the authors [A, 114, 116] to 'produce a simple calculation procedure which will, to a suitable degree of precision, estimate the temperature history in a fully developed compartment fire'. This model should take account of both kinds of fires (fuel and ventilation controlled).

It has been recognized that the major effect of fire on a building structure depends on the combination of fire intensity (maximum average air temperature) and fire duration [46, 73, 76]. It was expected that a general shape function in the parameters of the fire intensity and duration could describe the temperature history in a compartment fire. The physical parameters such as fire load and opening factor would only have an influence on the fire intensity and duration.

Based on the analysis of the measured fire curves for fully developed cellulosic fires, the shape function was given by

$$\frac{T_g - T_0}{T_{g\max} - T_0} = \left(\frac{t}{t_{\max}} \cdot \exp\left(1 - \frac{t}{t_{\max}} \right) \right)^{\delta} \tag{2.8}$$

where T_g and T_{gmax} are the air temperature and the maximum air temperature, respectively; T_0 is the room temperature; t and t_{max} are the time and the time when the maximum temperature is reached, respectively; δ is the shape constant of curve, usually 0.5 for the ascending phase and 1.0 for the descending phase; t_{max} is a function of fire duration τ. T_{gmax} and τ can be obtained from Eq. 2.1-2.6.

Comparisons with the measured curves showed that this model fits well for both fuel-controlled and ventilation-controlled fires.

2.3 Equivalent Fire Exposure

Due to the large variety of possible temperature-time curves in buildings, the assessment would be very expensive if the building elements were tested for each particular fire curve. Therefore, a nominal standard curve is necessary in some sense. The concept of equivalent fire exposure is a bridge between the realistic fire curve and the standard fire curve.

The equivalent fire exposure is the time during which a specified compartment or structure is submitted to ISO standard fire in order to obtain the same severity (effect) as the real fire curve. This concept has been developed into several different methods and the key difference lies in the definition of 'severity' or 'effect'. It is primarily intended as a measure for the rating of fire resistance for a compartment.

The development of equivalent fire exposure surrounds the selection of the 'indicator' for the equivalent, which represents the fire 'severity' or 'effect'. So far, there exist four indicators, including the area of fire curve, the maximum temperature of the critical part of the structures, load-bearing capacity and heat load.

2.3.1 *Equal Area Hypothesis*

The basic idea of the equivalent area hypothesis is that the area of the temperature-time curve under standard fire above a certain baseline (150 or 300 °C) corresponding to equivalent fire exposure should be equal to that under real fire.

According to this principle, Inberge obtained the first quantified relationship of equivalent fire exposure and fire load, by burning office furniture and papers in a room and measuring the temperature attained. Inberge also realized the importance of ventilation; he did not quantify its effects, but adjusted it to give what he deemed to be a severe condition. His work formed the starting point of the current regulations of fire gradings:

$$t_e = k_1 \cdot q \text{ (min)} \tag{2.9}$$

where q is the fire load density of equivalent woods per unit floor area (kg/m^2), and k_1 is the coefficient, which is of the order of unity.

However, Inberg's original hypothesis cannot be justified theoretically. Actually, the product of temperature and time is not the heat as expected. Because the heat transfer occurred during fire is dominated by radiative heat, the dependence of radiative flux on T_g^4 makes simple scaling very difficult (where T_g is the gas temperature of the fire compartment).

2.3.2 *Equal Maximum Temperature of Critical Part in Structural Elements*

Law (1973) [91] attempted to seek another way to express the severity of a compartment fire. She simplified the real fire as a constant temperature, which equalled the maximum air temperature, sustained for a period of time as fire duration (Eq. 2.6). Using the simplified natural fires, she analyzed the maximum temperature of insulated steel columns exposed to standard fire and real fires. The equivalent fire exposure was given by

$$t_e = k_2 \cdot q \cdot \frac{A_f}{\sqrt{A_w \sqrt{A_t - A_w}}} \text{ (min)} \tag{2.10}$$

where q is the fire load density per unit floor area (kg/m^2), k_2 is a coefficient.

Equation 2.10 can also be written as

$$t_e = c \cdot q \cdot w \tag{2.11}$$

where w is related to the ventilation conditions, and c is a constant coefficient. This type of equation has been extensively used by the Eurocode, CIB W14, Din 18 230 (1998) and the Code by New Zealand, except for using a different value of c and expressions for w.

2.3.3 *Equal Load-Bearing Capacity*

In this method, the residual sectional capacity of elements is taken as the equivalent indicator. Provided that the lowest sectional capacity under an overall fire process is R, then the time t_e corresponding to R under the standard fire curve, is defined as the equivalent fire exposure.

Based on this definition, the authors [109,114, 116] carried out extensive parametric studies on the RC columns, in which the parameters are the characteristic ratio of steel $\alpha = \rho f_y / f_{cm}$ (0.05-0.5), width of the cross-section *B* (0.3-0.6 m), the aspect ratio H/B (1-3), the thickness of protection *d* for reinforcements (10-70 mm). The following relationship was obtained

$$t_e = k_3 \cdot \sqrt{\tau \cdot T_{g\,max}^4} \tag{2.12}$$

where T_{gmax} is the maximum gas temperature (°C) and τ is the fire duration (min), both of which are ventilation and fire load dependent. k_3 is of the order of 8×10^{-6}.

The equivalent time by Eq. 2.12 is related only to the fire behaviour itself. In this equation, t_e is the function of the product of fire duration and $T_{g\,max}^4$, not the usual T_{gmax}. It is quite rational in such a case that the heat transfer under fire is dominated by radiation.

2.3.4 *Equal Heat Load*

Since the damage on the structures by fire is caused by the absorbed heat, it is also natural to use the heat load during the fire duration as an indicator. This idea has been developed since the earliest study, but was changed into the version of equal area hypothesis, for the purpose of simplicity. The author [109] made a study on the R.C. structures by equating the absorbed heat load under real fires and standard fire. The following equation was given

$$t_e = k_4 \cdot \left(\tau \cdot T_{g\,max}^3\right)^{2/3} \tag{2.13}$$

where k_4 is of the order of 4×10^{-6}.

Theoretically, Equation 2.13 is also suitable for composite structures and steelwork although they are developed from R.C. structures, because their basis is not strongly related with the materials of elements, according to the theory of NHL (Normalized Heat Load) uniformity [64,74]. The problem is that they are perhaps not suitable for exposed steelwork, since exposed steelwork is more sensitive to the maximum gas temperature than to the heat when the duration reaches a certain value.

2.3.5 *Equivalent Fire Exposure on Exposed Steelwork*

Since exposed steelwork is more sensitive to the maximum fire temperature than other building materials, the principle of 'equal maximum temperature of the critical part in structural members' (section 2.3.2) is reasonable to use. The method in Eurocode 1 Part 2.2 employs this principle. Figure 2.4(a) shows the comparison between numerical analysis [114] and direct calculation using Annex E of Eurocode 1 Part 2.2. In the numerical analysis, the parametric fire curves in Eurocode have been used as input. It can be seen that there is a large discrepancy. This reflects the inaccuracy in the current formulae for equal fire exposure on exposed steelwork in Eurocode.

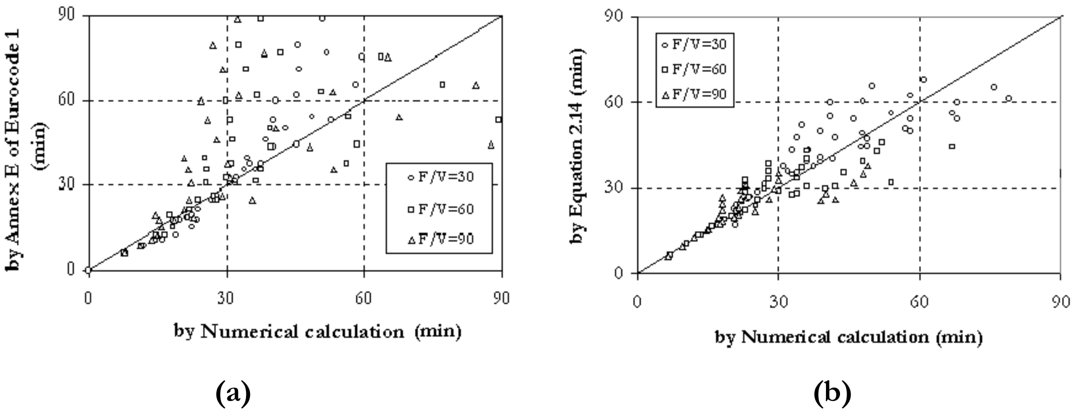


Fig. 2.5 Equivalent Fire Exposure for Exposed Steelwork - Comparisons between Equation Calculations and Numerical Analysis: (a) Eurocode Method; (b) Proposed Method by the Author [114].

A study by the author using the same principle has been carried out to obtain a high accuracy for exposed steelwork. In the study, the parametric fire curves according to Eq. 2.8 were used as the input of natural fire curves. The results showed that the index of the fire duration is section factor dependent. The following equation was given

$$t_e = k_5 \cdot \tau^{\alpha} \cdot T_{g\max}^{1.3} \tag{2.14}$$

where $\alpha = 0.267 + 5.87 \cdot \dfrac{V}{F}$, in which V/F is the inverse of the section factor; V is the volume of unit length, and F is the surface area of the steel of unit length. k_5 is of the order of 1.25×10^{-3}.

Equation 2.14 is quite similar to Eq. 2.12 in the case of a lower section factor. The equation here especially takes account of thin-walled steel members, whose inaccuracy in Eq. 2.12 is evident due to the coverage of the analyzed structures. On average, the resultant variable in Eq. 2.14 is the product of fire duration and $T_{g\max}^{b}$, where b is in between 2 – 4, dependent on the section factor. This indicates that for steelwork of a low section factor, the temperature of the steelwork is radiation-dominated, and for steelwork of a high section factor, the temperature is dependent both on the radiation and convection. Figure 2.5 illustrates the comparison between the analysis results and the results using Eq.2.14.

Chapter 3

Temperature Analysis of Composite Slim Floor Structures Exposed to Fire

3.1 Formulation of Heat Transfer

The transient heat transfer equation based on Fourier Law is given as

$$-\nabla^T(\lambda\nabla T)+\dot{e}=0 \tag{3.1}$$

where T is the temperature, λ is the thermal conductivity matrix, and $\dot{e}=de/dt$ is the rate of specific volumetric enthalpy change. The gradient operator ∇ is defined as

$$\nabla=\left[\frac{\partial}{\partial x}\ \frac{\partial}{\partial y}\ \frac{\partial}{\partial z}\right]^T \tag{3.2}$$

where x, y and z are Cartesian coordinates. Superscript T is the transfer operator of the vector.

In principle, the above equations can be derived as follows, by using the finite difference method

$$(\rho c+\rho_w c_w \varphi)\cdot\frac{dT}{dt}=\frac{T_1-T_0}{R_1\Delta x}+\frac{T_2-T_0}{R_2\Delta y}+\frac{T_3-T_0}{R_3\Delta x}+\frac{T_4-T_0}{R_4\Delta y} \tag{3.3}$$

where ρc is the volumetric specific heat, $\rho_w c_w$ is the volumetric specific heat of water, and φ is the volume ratio of moisture content. T_i is the temperature of the ith element in the local system (Fig. 3.1). R_i is the thermal resistance term, which can be given by

$$R_i=\frac{1}{K_i}+\frac{1}{K_0}+\frac{1}{K_{int}} \tag{3.4}$$

where K_i and K_0 are the thermal conduction terms. K_{int} is the interface resistance coefficient. The sensitivity analysis in [110] has shown that the value $K_{int}=50\,W/m^2K$ could be used for the interface coefficient between steel and concrete. In the case of perfect contact. $K_{\text{int}}=\infty$ can be presumed.

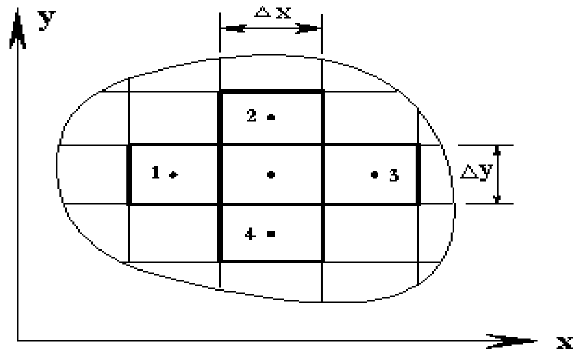


Fig. 3.1 Local Mesh of Control Volume for Finite Difference Analysis.

A generalization of Eq. 3.3 for the finite difference analysis was presented in [D]. Besides, Eq. 3.3 was derived for the internal elements. The complete formulation of heat transfer also includes the equations for the boundary elements and the auxiliary equations [D, 110].

The heat input rate q is expressed as

$$q = K \cdot (T_g - T_s) \tag{3.5}$$

where T_g is the air temperature of fire and T_s is the surface temperature of the structures. K is the coefficient of total heat transfer. According to ECCS Recommendations [26], it can be determined by

$$\frac{1}{K} = \frac{1}{\alpha} + \frac{d}{\lambda} \tag{3.6}$$

where d is the thickness of insulation. λ is the conductivity of insulation material; and α is the resultant convective factor, which can be given by

$$\alpha = \alpha_c + \alpha_r = \alpha_c + \sigma\varepsilon \cdot \left[(T_g + 273)^4 - (T_s + 273)^4\right] / (T_g - T_s) \tag{3.7}$$

where α_c is the coefficient of heat transfer due to convection from fire to the exposed surface of the member; α_r is the coefficient of heat transfer due to radiation; σ is the Stephan-Boltzmann constant, and ε is the resultant emissivity coefficient.

The effect of moisture in concrete was modelled by the modified specific heat of concrete before its temperature reaches to 120 °C. After that the absorbed heat is totally used to evaporate the water until the water content of the element becomes zero. The formulation of the volume change of water within the elements has been given in [D, 110].

3.2 Computer Implementation

According to the above formulation of heat transfer, a temperature analysis program (TACS-FIR) for composite structures using the finite difference method was written by the authors [110]. The temperature distribution and history of both composite and concrete structures can be analyzed. The fire curves can be ISO standard fire or any type of natural fires. The program has been validated by the fire tests on composite slim floor beams (280ASB and 300ASB) by BRI [95], concrete-filled tubular columns by VTT [137] and concrete columns by HUCEA [121]. A comparison of the analysis results between this program and the TASEF commercial software was also made. Using this program, the temperature of composite structures in fire can be evaluated without the necessity of testing.

The computer program was coded in Fortran 77. The geometrical divisions of the current version are limited to the rectangular grids. The material thermal properties at elevated temperature can be selected by the user inputting the material ID according to the different codes or researchers, which were summarized in [110]. The material properties can also be input in the form of differentiated data defined by the user. In that case it is assumed to vary piece-wise linearly with temperature between the neighbouring data points. The conductivity of concrete can be specified to remain approximately as that at maximum temperature instead of the current temperature in the cooling phase. The critical time increment is usually very short, which is mainly dependent on the minimum dimension of the cells in the steel web. The theoretical criterion was derived in [D, 110]. A warning message will occur and the proposed value will be given on the screen if the inappropriate time increment was given.

The fire temperature history may be defined as a number of differentiated points on a temperature-time curve. Temperature between these points is obtained by internal interpolation. Several fire models described in Chapter 2, which are of the mathematical expression, are provided by the program and can easily be selected by inputting the according ID. These fire models are ISO standard fire (ID=2), parametric fire curves according to Annex B of Eurocode 1 Part 2.2 (ID=3) and the proposed fire model by the authors (ID=4).

The explicit forward difference scheme described in the previous section is used. The temperature at the central point represents the temperature of the elements. The maximum temperature of each element is specially stored in a vector for use in calculating the material properties in the cooling phase if necessary (for concrete only). The element temperature is output at the specified times.

The major structure of this program is illustrated in Fig.3.2 and the major routines are shown in Table 3.1. The invoking relationship of these routines and the main variables are described in [110].

TABLE 3.1 Functions of Subroutines in TACS-FIR.

No.	*Subroutines*	*Description of functions*
1	INPUT	Input the geometric, physical and other parameters of the object for analysis
2	FIR_CAS	Handle the fire temperature-time curves. Including a small collections of several fire curves
3	TEMPER	Analyze the temperature of internal elements
4	THER_RES	Calculate the thermal resistance items of internal elements
5	MATERIAL	Calculate the thermal properties of steel and concrete at elevated temperature
6	WATER	Calculate the evaporation of free water content in concrete for internal elements
7	BOUNDARY	Analyze the temperature of boundary elements and handle all boundary conditions
8	B_RES	Calculate the thermal resistance items of boundary elements
9	B_WATER	Calculate the evaporation of free water content in concrete of boundary elements

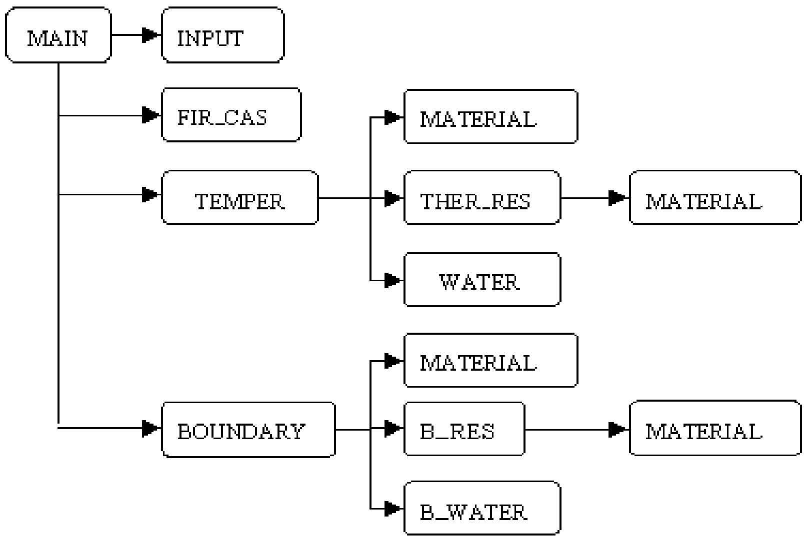


Fig. 3.2 Structure of the Computer Program TACS-FIR [110].

3.3 Thermal Response of Composite Slim Floor Beam to Fire

Figure 3.3 shows the temperature distribution of the Finnish composite slim floor beam with and without fire protection underneath the bottom steel flange at 60 minutes' fire exposure.

49	48	47	44	40	35	30	28	29	31	33	34
57	56	55	52	46	39	33	30	31	34	37	39
77	77	74	70	61	50	39	34	38	42	48	51
113	112	109	102	89	68	50	41	45	49	57	65
172	170	166	156	134	99	67	48	44	48	62	80
265	263	258	244	210	146	91	59	47	51	73	97
418	416	411	393	341	218	124	74	54	58	86	117
687	685	681	663	594	325	165	92	64	67	101	140
					481	210	111	76	79	117	169
					563	246	131	90	91	125	203
					605	274	151	107	107	142	242
					631	299	174	130	130	167	287
					653	328	206	162	162	201	338
					679	369	253	211	211	250	398
					714	432	327	288	285	320	469
					763	535	447	410	401	423	552
					864	834	803	770	735	701	649

(a)

49	48	47	44	40	34	29	26	24	25	25	25
57	56	54	51	46	38	32	27	26	26	27	27
77	76	74	69	61	49	38	31	29	29	31	32
113	112	109	102	88	68	49	37	35	36	39	41
172	170	166	156	134	98	66	45	40	41	46	51
265	264	258	244	210	145	90	56	42	41	49	60
417	416	410	392	341	217	123	71	48	45	56	71
686	685	680	662	593	323	163	88	56	51	65	84
					479	207	106	66	59	76	99
					560	241	123	77	69	89	117
					600	265	139	89	81	105	138
					622	285	155	104	95	120	164
					636	303	174	124	114	133	194
					649	325	201	151	140	156	229
					659	356	241	193	178	190	269
					663	406	305	258	237	240	314
					625	571	522	478	437	396	365

(b)

Fig. 3.3 Temperature Distribution of Composite Slim Floor Beam at 60 minutes' ISO Fire Exposure: (a) without Fire Protection; (b) with Fire Protection (intumescent fire paint of 250 μm).

Chapter 4

Structural Behaviour of Slim Floor Construction in Fire

4.1 Introduction

When subjected to fire, steel and composite structures lose their loading capacity and stiffness. To ensure safety to the life and prevent property loss, the indispensable fire resistance of the building is required by the authorities [36]. Traditionally, the fire resistance ability of the structural members was tested using the isolated element heated by the ISO standard fire. In this methodology, buildings were treated as a series of individual members, and the continuities and interaction between these members were assumed to be negligible. Consequently, most of the structural members need to be protected by the insulation materials, such as intumescent paints and fire protection boards, in order to achieve the required fire resistance.

Throughout the 1990s, following the investigation of the fire event in Broadgate (1991, UK), fire tests in William Street (1992, Australia), and full-scale fire tests on an 8-storey composite steel-framed building in Cardington (1995, 1996, UK) [79,162], it was found that the structural member in the frame exhibited significantly better behaviour in fire than that in the standard fire-resistance test. The standard fire test was very conservative by disregarding the interaction between members. The fire event and tests also highlighted that the current Codes, although conservative, did not address the true behaviour of the building structure in fire, since the building was not acting as a series of individual members [8-9, 52-53, 131, 151].

In recent years, increasing interest has also been shown throughout Europe to develop and design a shallow floor system in steel-framed buildings [95, 128, B, C, 110, 111, 112]. In the shallow floor system, the steel beam is contained within the depth of the pre-cast concrete floor or composite slab with profiled steel decks. Recently, interest has been concentrated on asymmetric hot-rolled steel in UK [130] and on the asymmetric welded steel beam in Finland [B, C, 110-113].

The aim of this research is to investigate the fire resistant behaviour of the Finnish asymmetric slim floor beam both as an isolated member and as a part of frame. The research results have been published in [B, C, 110-112]. The thermal analysis of the composite slim floor beam was included in publications [D, 110]. The fire resistance of the composite slim floor beam as an isolated member, with varied enhanced measures such as fire protection additional reinforcements, was described in publications [B, C, 110, 111]. The effect of various axial restraints on the fire resistance of isolated members

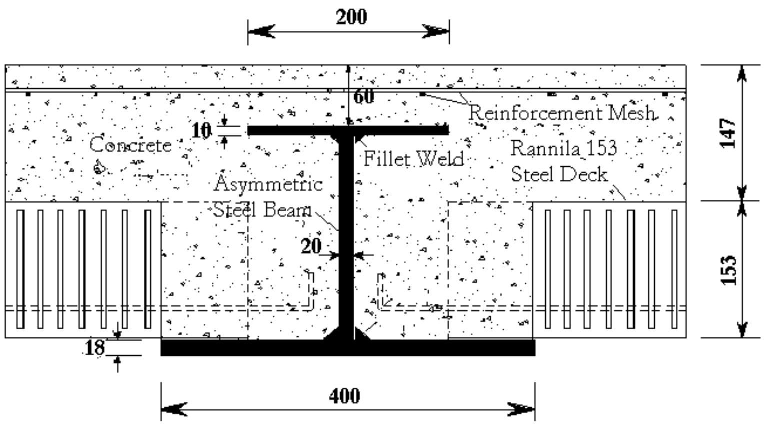


Fig. 4.1 Section Shape of the Composite Finnish Asymmetric Slim Floor Beam.

has been published in [111]. Effects of the axial and rotational restraints in two-dimensional subframes and the effects of composite floor slab were preliminarily studied in [C, 111]. Both the deformation and mechanical responses to fire have been explored.

For further research on the behaviour of the slim floor frame in fire, the overview in this context (Section 4.4) concentrates on the latest research results, which have not yet been published due to time issues. The content includes the global and local deformation response of the whole frame to fire, and the force and moment distribution and variation with time of the frame members. The mechanism of the beneficial and detrimental effects of the slim floor beam as a part of frame was explored. The effect of columns being protected and unprotected, and effect of a sway and non-sway frame were also investigated. The effect of the composite floor slab was further studied by three-dimensional frame analysis.

Most of the research conclusions for the frame analysis were also applicable for the other types of composite frame structures in fire conditions, even though the analyses were made on the composite slim floor structures in this context.

4.2 Fire Resistance Analysis of Isolated Beams - Eurocode Methods

Figure 4.1 shows the section shape of the Finnish asymmetric slim floor beam. The slim floor steel beam consists of three plates welded together. The steel deck is a Finnish product, Rannila 153.

Using the temperature distribution of the beam section in fire described in Chapter 3, the plastic moment capacity of the slim floor beam at elevated temperatures was analyzed by the moment capacity method. The mechanical properties of steel and concrete under high temperatures are given according to Eurocode 3 and Eurocode 4. The studies have shown that the composite slim floor beam has a 60-minute' fire resistance if the load ratio is lower than 0.47. In practice, the load ratio of 0.5-0.55 is the most common when an office or residential building is subjected to fire. To achieve the desired fire resistance, some enhanced measures such as additional reinforcements and fire protection on the underside of the bottom steel flange must be taken. The 60-minute' fire resistance can be obtained if rebars 2d32 are added in the concrete close to the bottom flange and the according load ratio is 0.57. Fire protection such as insulation board or intumescent fire paint underneath the bottom steel flange is very efficient. When the intumescent fire paint Nullifire® and an allowed minimum thickness of 250 μm are used, the fire resistance can be up to 90 minutes under the load ratio of 0.67.

Resistance analysis under natural fires according to Eurocode 1 Part 1.2 was also made in [111]. The critical load ratios were given for different natural fire curves, which were in the parameters of opening factor and fire load density. After the analysis, it was concluded that the composite slim floor beam could be used without additional measures if the fire load density of the apartment is lower than 1100 MJ/m^2, which is rarely exceeded for office and residential buildings.

The contributions from each part of the composite beam section have also been analyzed. In the analysis, the beam section was divided into five parts: bottom steel flange, lower part of steel web, upper part of steel web, top steel flange and compressive concrete slab. The contribution analysis showed that at 60 minutes' fire exposure, the moment capacity contribution of the bottom flange is reduced to 31% from the original value of 44% at ambient temperature. The lower part of the web contributes 53% of the total moment capacity at 60 minutes' fire exposure while this value is 6% at ambient temperature. The steel web contributes a major part to the total moment capacity in fire while the bottom flange has a major contribution at ambient temperature.

The fire resistance of the composite slim floor beam using a hybrid steel beam was studied in [112]. In the hybrid steel beam, the bottom steel flange was substituted by high-strength steel or fire-resistant steel to achieve the desired behaviour of the composite beam.

4.3 Numerical Analysis of Isolated Beams

The numerical analysis of the beams at elevated temperature was performed using the general finite element program, ABAQUS/Standard [1]. The material properties of steel and concrete were taken from Eurocode 4 Part 1.2. The temperature distribution and history described in Chapter 3 were used. Uniformly four-point loading was applied and the corresponding load ratio was varied from 0.35 – 0.7. The parameters included additional reinforcements, fire protection on the bottom steel flange, beam spans and axial restraint stiffeners.

The displacement response of the beam to fire was investigated. First of all, two fire tests carried out by Warrington Fire Research Center (WFRC) [95] were analyzed, in order to validate the finite element model. Then, the parametric analysis was carried out. The displacement response of the Finnish slim floor beam under an applied load ratio of 0.53 is shown in Fig. 4.2.

The parametric analysis has shown that a 60-minute' fire resistance can be achieved if the load ratio is lower than 0.53. Higher fire resistance can be obtained by reducing the load ratio, employing additional reinforcements or fire protection on the bottom

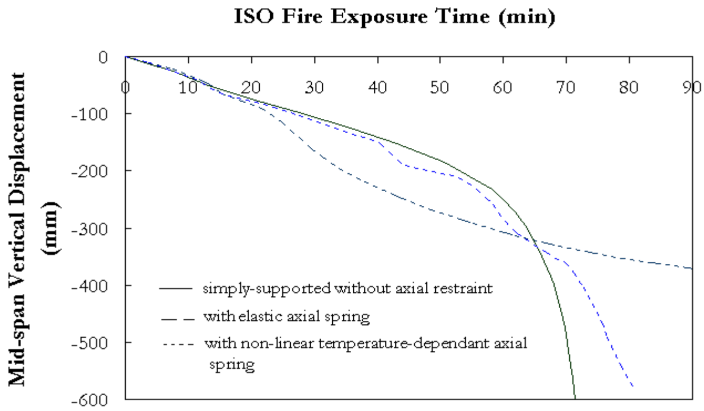


Fig. 4.2 Displacement Response of the Composite Slim Floor Beam under ISO Fire.

flange. Intumescent fire paint of 250 μm can increase the fire resistance up to 105 minutes when the applied load ratio is 0.53. The studies also showed that the effect of the beam span on fire resistance is negligible if the applied load ratios are the same. The beams with constant axial restraints have a larger displacement at the earlier phase of ISO fire due to the secondary P-Δ effect caused by the thermal expansion. However, a more stable deformation response can be achieved thereafter (see Fig. 4.2). If the non-linear force-displacement curves with varied temperature are used for the axial springs, the deformation curve of the beam is in between that with constant stiffness and that without axial restraints.

4.4 Numerical Analysis of Frames

Figure 4.3 shows a typical frame structure using a composite slim floor system. Three types of frames, which were extracted from this structure, were analyzed. These frames include one-bay two-storey 2-d subframes, two-bay four-storey 2-d frames and two-bay two-span two-storey 3-d frames. The parametric studies on the 2-d subframes aimed to investigate the effect of semi-rigid connection (rotational restraints) and its quantification, the effect of the axial restraint provided by the columns, and the axial force and moment variation of the beams and columns in fire. In the study, the steel columns were filled in between the steel flanges with the aerated concrete.

Analysis of the 2-d five-storey frames aimed to investigate the structural behaviour of the composite slim floor frame as a whole in fire conditions. Both the deformation behaviour of the structural members and the mechanical interaction between the members were studied. The additional lateral deformation of the side-column caused by the thermal expansion and the catenary action in the beam in the different fire phases was highlighted. The moment variation in the columns during fire and the variation of the axial force in the heated beam were also investigated.

Analysis of the 3-d frames aimed to investigate the effect of composite floor slab on the behaviour of frame in fire. Comparison between the deformation behaviour of the heated beam in the plane frame and the spatial frame indicated the excellent effects of the concrete floor slab on the stability of the frame structures in fire.

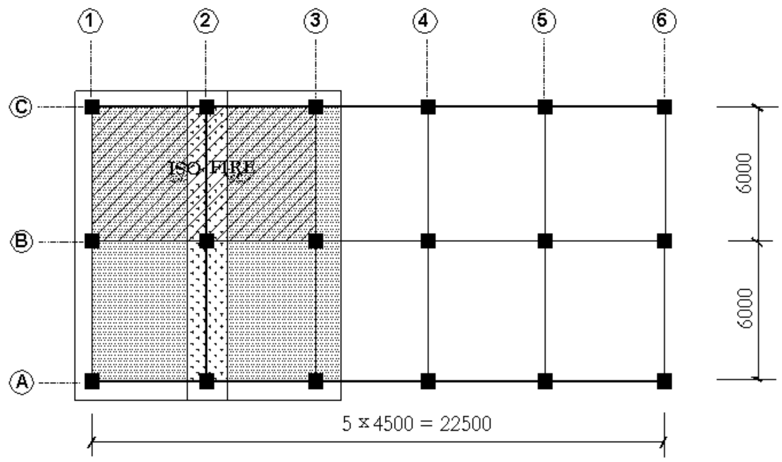


Fig. 4.3 Layout of the Analyzed Frame Building using Composite Slim Floor System.

4.4.1 *Effects of Axial and Rotational Restraints on Beams*

The structural behaviour of a beam or column in a plane frame can be resolved into an individual member with both rotational and axial restraints at two ends, as shown in Fig. 4.4. In the figure, K_r, K_a and K_v are the stiffness of the rotational, axial and support restraints respectively, provided by the surrounding columns and beams. Usually, the support restraint can be simplified as rigid. In a 3-d frame, the restraint caused by the floor slab should be included, on condition that the floor slab was compositely connected with the beams.

In [C], a method employing the 'modified load ratio' was put forward to quantify the effect of rotational restraint on the fire-resistant behaviour of the beam. The method was generalized to consider the rotational restraints provided by the semi-rigid column-beam connections.

Two major schemes have been carried out to investigate the effect of axial restraint on the fire-resistant behaviour of the beam. In the first scheme, the individual beams were subjected to fire with axial restraint spring of linear elastic properties and nonlinear temperature-dependent properties. Only the deformation behaviour of the beam in fire was investigated in this scheme. In the second scheme, the beam in the frame with hinged connection, was subjected to fire. Both the deformation and mechanical responses were analyzed in this scheme.

The effect of axial restraint on the mid-span vertical displacement for a simply supported beam in fire is shown in Fig. 4.2. It can be seen that the beam with axial restraint achieves a stable deformation response in the later phase and the run-away point is significantly detained, whereas before that, it causes larger deformation than without axial restraints. The mechanism is interpreted later.

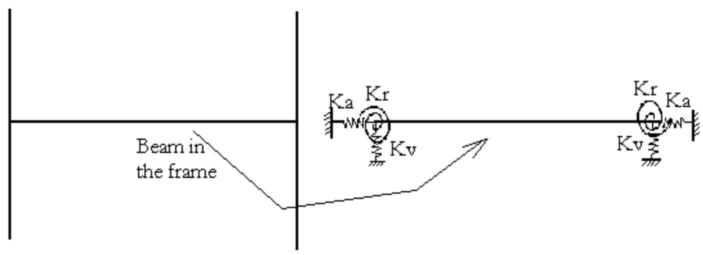


Fig. 4.4 Restraints of the Beam in a Frame Structure.

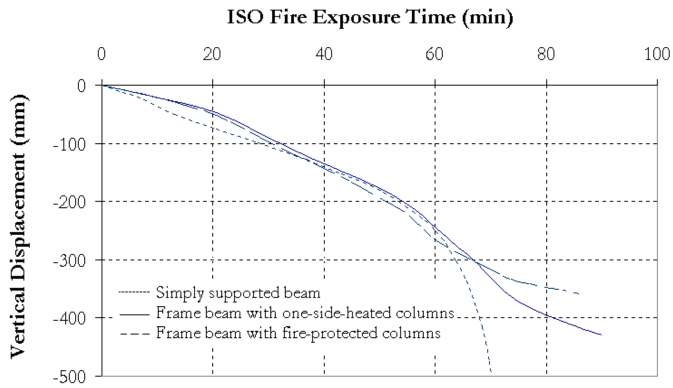


Fig. 4.5 Vertical Mid-span Displacement of the Heated Beam with Hinged Beam-Column Connections.

For the cases in the second scheme, the deformation responses of the beam were similar to the results in the first scheme (see Fig. 4.5). In the figure, the columns in the frame were left unprotected. The according analysis results have been described in detail in [115]. Figure 4.6 shows the axial force variation of the heated beam during the fire exposure. It can be seen that the axial force in the beam is compressive and increases rapidly at the earlier ISO heating phase, which is due to the axial restraints by the surrounding structure on the thermal expansion of the beam. However, with the increasing deformation of the beam, catenary action comes into effect. Meanwhile, the heating rate of the beam becomes slow with time and consequently the catenary action occupies the leading role. At around 60 minutes' fire exposure, a transient balance between the thermal expansion and catenary action occurs, and the axial force becomes zero. After that, the beam starts to be in tension. Another important factor, which contributes to the development of the axial force in the beam, is the decreased restraints by the column due to the material softness of steel at elevated temperature. This point is clearly reflected by the comparison between the column being heavily protected or unprotected, which is highlighted in Fig. 4.6.

An interesting point can be seen by looking through Figs. 4.5 and 4.6. The frame beam still maintains its stability at 62 minutes' fire exposure whilst the simply supported beam starts to run away. This corresponds to the time that the axial force in the beam starts to become tensile (Fig. 4.6). The catenary action starts to take effects to maintain the stability of the frame beam when the simply supported beam starts to run away under the modified load ratio.

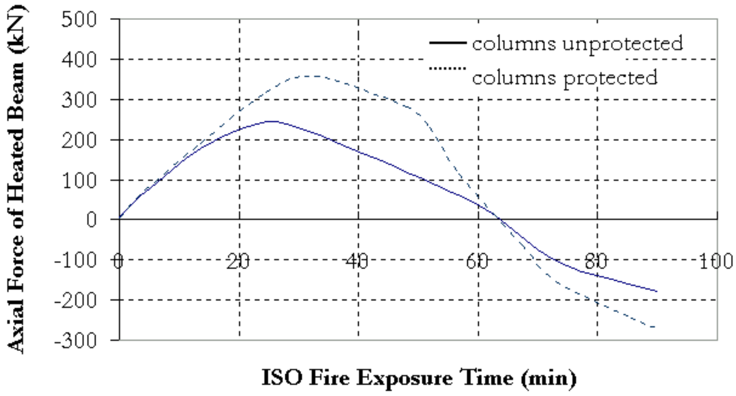


Fig. 4.6 Axial Force-time Curves of the Slim Floor Beam in the Frame under ISO Fire.

4.4.2 ***Effects of Fire Protection on Columns and Heat Rate on the Beam***

In some practices, the columns are fire protected to avoid their possible collapse during fire. In fact, fire-protected columns provide a larger restraint on the heated beam in the course of fire attack. In publication [115], the effect of fire protection measures on the column are presented. The studies showed that the fire protection measures produced better deformation behaviour for the heated beam and a smaller lateral displacement in the column end after 60 minutes' ISO fire exposure. However, the protection measures also caused a bigger axial force (both compressive and tensile) in the beam throughout the fire exposure.

In reality, the heating rate of the structural members may be different in each fire event, because of the different fire protection on the members or the different fuel types in the fire room, and so on. Three kinds of heating rates for the bottom flange of the slim floor beam were used in this study. Case 1 represents a linear heating rate of the bottom flange. Case 2 represents the heating rate caused by the ISO standard fire curve, and Case 3 represents a faster temperature rise in the earlier stage of fire. For all cases, the bottom steel flanges reach the same temperature at 90 minutes.

The analysis results show that the structural responses of the frame corresponding to the same temperature of the bottom steel flange are very close to each other, if the columns in the analyzed frames are heavily protected. However, if the heated columns are unprotected, the differences between the different heating rates are rather significant. This can be attributed to the interaction between the beam and the surrounding columns in fire. The analysis results are listed in Table 4.1. Actually, corresponding to the same temperature in the bottom flange of the beam, the fire exposure time is different for each of the different cases of heating rate. The according temperature distributions in the columns are hence different, which cause the different axial constraints on the heated beam.

Table 4.1 Structural response to different heating rates - exposed columns

Heating rate		*Beam mid-span displacement (mm)*	*Axial force in beam (kN)*	*Lateral displacement of side-column (1st floor) (mm)*	*Additional moment of side-column (kNm)*
T=600 °C	Case 1	-140	104	-12.8	126
	Case 2	-153	164	-12.2	248
	Case 3	-192	226	-9.8	407
T=890 °C	Case 1	-418	-193	22.0	-163
	Case 2	-432	-177	22.1	-130
	Case 3	-454	-156	32.9	-82

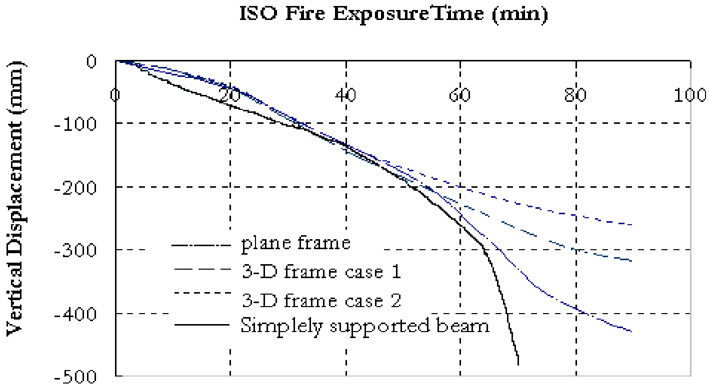


Fig. 4.7 Vertical Displacement of the Heated Beam in Plane Frame and 3-D Frame. (In the case 1 of 3-D frame, the beams along axis 1 and 3 were assumed to be well-protected and only the beam along axis 2 was heated; in the case 2, the beams along axis 1, 2 and 3 were heated. It was assumed that the columns C/2 and B/2 were subjected to fire.)

4.4.3 *Effects of Composite Floor Slab*

In order to identify the influence of the composite floor slab on the fire-resistant behaviour of the heated beam and the whole frame, several three-dimensional frames were analyzed. In the analyzed frames, the beam was connected by a hinge to the columns so that the rotational restraints were removed and only the axial restraints and the restraint of the composite floor slab existed. Then, the deformation and mechanical responses of the three-dimensional frame were compared with the plane frame which was similar to the three-dimensional frame but only the composite slab of a width of 1/8 beam span was included. In this way, the influence of the composite floor slab was expected to be identified.

Figure 4.7 shows the deformation response of the heated beams in an isolated case, in a plane frame and in a three-dimensional frame. It can be seen that the beam in the three- dimensional frame has a significantly better deformation behaviour after 60 minutes' fire exposure, whereas the deformation curves of the beams in the three-dimensional frame and plane frame are very close to each other before that. In this figure, the deformation response of the heated beam with the surrounding beams also heated, was shown. When the surrounding beams were also heated, the same as the central beam, the restraint on the composite floor slab was weakened and the heated central beam had a larger displacement.

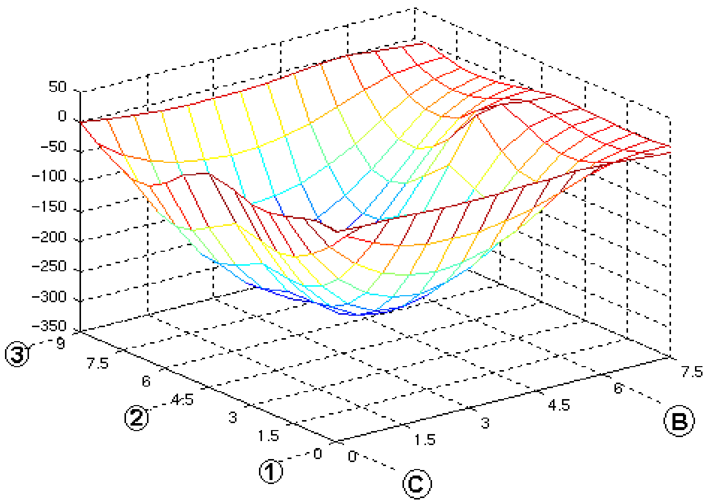


Fig. 4.8 Deformation Profile of Floor Slab at 90 minutes' ISO Fire Exposure.

Figure 4.8 shows the deformation profile of the floor slab at 90 minutes' ISO fire exposure. It can be seen that the floor slab seems to support the beam, not the beam supporting the floor slab. This phenomenon is similar to that in the Cardington fire tests, which can be attributed to the 'membrane action' in the floor slab that becomes significant in the case of large displacement.

4.4.4 *Effects of Sway and Non-sway Frame*

For a sway frame, research has shown that the frame collapsed at a very early phase of ISO fire exposure (20 minutes in the studied case), if the columns were left unprotected. This was mainly caused by the significant P-δ effect and the large thermal expansion of the heated beam. However, if the columns were fire-protected, the frame kept its stability up to 90 minutes' fire exposure, which was close to the non-sway frame.

4.4.5 *Global Deformational and Mechanical Responses of the Frame in Fire*

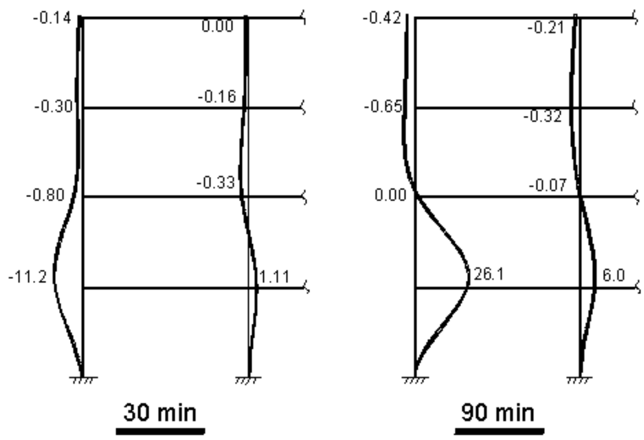
The global deformation of the frame is shown in Fig. 4.10. As interpreted later, the lateral deformation of the frame is largely caused by the variation of axial force in the heated beam. In the earlier phase, the thermal expansion of the heated beam produced a push to the surrounding structures. In the later phase, the catenary action took effect and a pull to the surrounding structures was produced. The lateral deformation of the frame follows the variation of the axial force within the heated beam.

Study of the mechanical response to fire of the frame members lends itself to understanding the mechanisms behind the deformation response. Studies in this scheme showed that three kinds of restraints had the major influence on the behaviour of the frame structure in fire, which are axial restraints, rotational restraints and the restraints by the composite floor slab. The later, causes a phenomenon that has been described as 'the floor slab supports the beam' in fire, which is actually produced by the catenary action in the slab in case of large deformation. In this scheme, the plan frame was investigated and the variation of the axial force in the heated beam and the moment variation in the columns were tracked. The research results are presented in publications [B,C,111, 115].

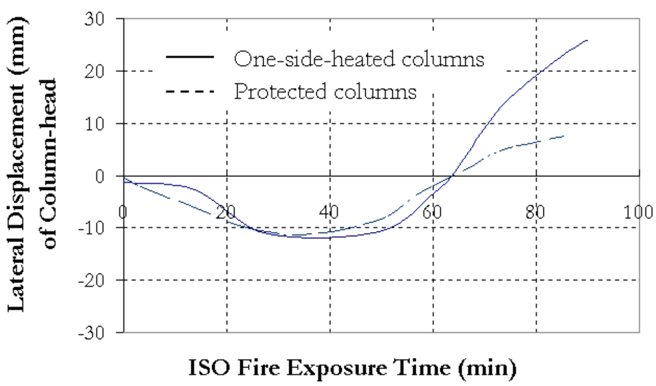
Generally, the variation of the axial force in the heated beam can be divided into two phases (see Fig. 4.6). At the early ISO fire exposure, the axial force is compressive and increases rapidly up to several times of that before heating. This is attributed to the axial restraints caused by the surrounding structures on the thermal expansion of the beam. However, with the increasing deformation of the beam, the catenary action starts to take effects and gradually takes the leading role. This is the second phase. After a period of time, a transient balance between the thermal expansion and catenary action takes place, and the axial force becomes zero. After that, the beam starts to be in tension.

All factors that affect the degree of the axial restraints in the heated beam contributes to the development of the axial force in the beam, for example, the material softness of steel columns at elevated temperature. A comparison of the axial force response between the frames with the columns being fire-exposed and fire-protected is shown in [115].

The variation of the axial force in the heated beam causes the significant additional moment in the surrounding frame members, especially in the columns. Fig. 4.11 illustrates the moment distribution of the columns at 30 and 90 minutes' fire exposure, respectively. In normal conditions, the moments in the columns are very small for the studied frame. It can be seen that the additional moments in the columns under fire are surprisingly large.

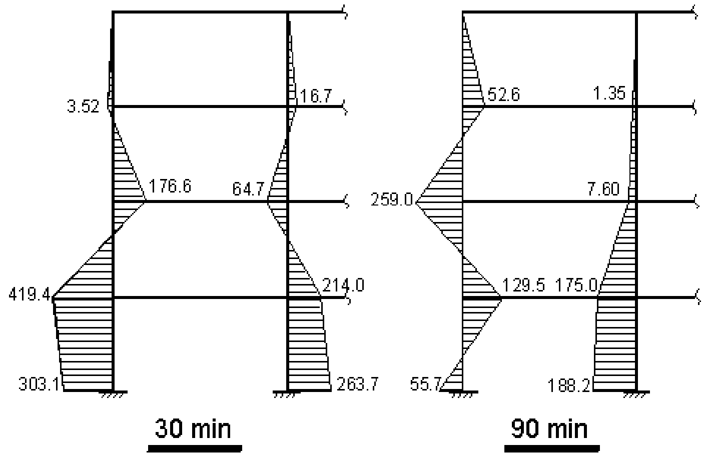


(a)

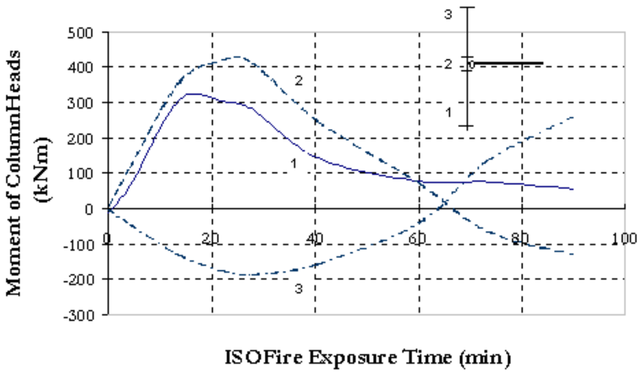


(b)

Fig. 4.10 Global Deformation of the Frame in Fire: (a) Deformation profile of the frame at 30 and 90 minutes' fire exposure; (b) Lateral displacement – time curves of the side – column C/2 in the first storey.



(a)



(b)

Fig. 4.11 Moment Distribution and Variation with Time of the Columns in Fire: (a) Moment distribution at 30 and 90 minutes' fire exposure; (b) Moment variation with time of the side-column C/2 in the 1st and 2nd storey.

Chapter 5

Conclusions

Structural fire safety design is an important aspect in the design of steel and composite steel-concrete structures. Development of the structural aspect of modern fire safety design includes the characterization of fully developed fire curves, the temperature analysis and the structural response analysis. The probabilistic aspect of the fire curves and the action on the structures in fire are also very important issues. The probabilistic aspect of the fire curves could be considered using the probabilistic fire load and ventilation, in which the effects of sprinklers and fire-fighting by the fire brigades can also be included in a convenient way. The action (applied loads) on the structures in fire was developed by survey and probability analysis [42]. This dissertation is concerned with the characterization of fully-developed fire curves, the temperature analysis and the structural response analysis.

A method to estimate the temperature-time curve in a fully developed fire compartment for structural design purposes, without using computer code, has been developed, on the basis of extensive test results and increasing knowledge of fire engineering science. This method can provide a quick calculation of natural fire curves for use in the structural fire-resistance design, with a rational precision. Accordingly, the equivalent fire exposure methods have been developed. The equivalent fire exposure was found to be in the function of the product of fire duration τ and the maximum gas temperature $T^{b}_{gmax,}$ where b is in between 2 – 4, according to the equivalent principle. This composition indicates the fact that fire is radiation-dominated.

A heat transfer model has been developed to analyze the temperature distribution and variation with time when the composite structures are exposed to fire. In the model, the effect of moisture in the concrete and fire protection on the steel beam can be taken into account. A computer code using the finite difference method (TACS-FIR) has been written.

In recent years, increasing interest has been shown throughout Europe to develop and design slim floor systems in steel-framed buildings. This dissertation presents the fire resistance behaviour of the composite asymmetric slim floor beam both as an isolated member and as a part of the frame using numerical analysis methods. In general, three schemes have been investigated, including isolated beams, plane frames with semi-rigid and hinged beam-to-column connections and the three-dimensional slim-floor frame system. The first scheme aimed to explore the fire resistance of the beams according to standard fire-testing methodology. The objective of the second scheme was to reveal the effects of frame continuity on the fire resistance of the slim floor beam and the mechanical interaction between the frame elements. The third scheme aimed to identify the influence of the composite slab on the beam behaviour in fire. The analyzed results indicate that the axial restraints provided by the surrounding parts cause a major influence on the global deformational and mechanical responses of the frame to fire. The rotational restraints essentially cause the change in the applied load ratio in fire, which can be quantified using the 'modified load ratio' proposed in the context.

Bibliography

[1] ABAQUS/Standard. User's Manual (version 5.7). Hibbitt, Karlsson & Sorensen Inc., 1997.

[2] Ali, F.A., Shepherd, P., Randall, M., Simms, I.W., O'Connor, D.J. and Burgess, I. The Effect of Axial Restraint on the Fire Resistance of Steel Columns. Journal of Constructional Steel Research 1998; 46 (1-3): Paper No.177.

[3] Babrauskas, V. and Williamson, R.B. Post-flashover Compartment Fires, Fire Research Group Report No. UCB FRG 75-1, University of California, Berkeley, 1975.

[4] Babrauskas, V. and Williamson, R.B. The Historical Basis of Fire Resistance Testing, part I. Fire Technology 1978; 14 (3): 184-194.

[5] Babrauskas, V. and Williamson, R.B. The Historical Basis of Fire Resistance Testing, part II. Fire Technology 1978; 14 (4): 304-316.

[6] Baily, C.G. The Behaviour of Asymmetric Slim Floor Steel Beams in Fire. Journal of Constructional Steel Research 1999; 50: 235-257.

[7] Baily, C.G., Burgess, I. and Plank, R. Bridging and Restraint Effects of Localised Fires in Composite Frame Structures." Proc. of the International Conf. Composite Construction, Innsbruck, Austria, 1997: 379-384.

[8] Baily, C.G., Lennon, T. and Moore, D.B. The Behaviour of Full-Scale Steel-Framed Building Subjected to Compartment Fires. The Structural Engineer 1999; 77 (8): 15-21.

[9] Bailey, C. Computer Modelling of the Corner Compartment Fire Test on the Large-Scale Cardington Test Frame. Journal of Constructional Steel Research 1998; 48: 27-45.

[10] Baldwin, R. and Nonth, M.A. The Number of Sprinkler Heads Opening in Fires. Joint Fire research Organisation Fire Research Note 886, Borehamwood, 1971.

[11] Babrauskas, V. A Close-form Approximation for Post-flashover Compartment Fire Temperature. Fire Safety Journal 1981; 4: 63-73.

[12] Barnett, J.R. Structures Subjected to Accidental Fires. http://www.wpi.edu/Academics/Depts/Fire/ papers/chicago/chicago.html. WPI Center for Firesafety Studies, Worcester, USA, 1995.

[13] Becker, W. and Hertel, H. Fire Safety Concept. Fire Safety Journal 1991; 17 (2): 103-111.

[14] Bennetts, I.D., Proe, D.J., Lewins, R.R and Thomas, I.R. Open-deck Car-park Fire Tests. BHP Melbourne Research Laboratories, Report MRL/PS69/85/001, Australia, 1985.

[15] Bennetts, I.D., Proe, D.J., Lewins, R.R and Thomas, I.R. Fire and Unprotected Steel in Closed Car-parks. BHP Melbourne Research Laboratories, Report MRL/PS98/87/001, Australia, 1987

[16] Bernuzzi, C., Gadotti, F. and Zandonini, R. Semi-continuity in Slim Floor Steel-Concrete Composite Systems. 1st European Conference on Steel Structures, Athens, Greece, May 18-20, 1995.

[17] Bernuzzi, C., Gadotti, F. and Zandonini, R. Joint Action in Slim Floor Systems, PSSC´95, 4th Pacific Structural Steel Conference, Volume 3: Steel-Concrete Composite Structures, Singapore, 1995.

[18] Björkman, J. and Keski-Rahkonen, O. Simulation of the Steckler Room Fire Experiment using SOFIE CFD-model. Research Center of Finland, VTT publications 265, 1996.

[19] Black, N.S. Ultimate Strength Study of Two-way Concrete Slabs. Journal of Structural Division, Proceedings of American Society of Civil Engineers 1975; 101 (ST1): 311-324.

[20] Bohm, B. Calculated Thermal Exposure of Steel Structures in Fire Test Furnaces – a non-ε_{res} approach. Laboratory of Heating and Air Conditioning, Tech. University Denmark, CIB W14/82/73 (DK).

[21] Bolwin, R., Law, M. and Giffiths, L.G. Survey of Fire Loads in Modern Office Buildings- Some Preliminary Results. Fire Research Note No.808, Joint Fire Research Organization, England, 1970.

[22] Both, C., Fellinger, J.H.H. and Twilt, L. Shallow Floor Construction with Deep Composite Deck: From Fire Tests to Simple Calculation Rules. Heron 1997; 42 (3): 145-158.

[23] Both, C. Fire-Exposed Continuous Span Composite Steel- Concrete Slabs. Heron 1996; 41 (3): 187-199.

[24] Both, C., Twilt, L. and Stark, J.W.B. Numerical Simulation of Thermal and Structural Response of Composite Steel/Concrete Slabs to Fire. 4th Pacific Structural Conference, Singapore, Volume 3, October 1995.

[25] Both, C. and Twilt, L. The Real Behaviour of Modern Largely Unprotected Steel Framed Buildings under Natural Fire Conditions. Diana World 1997; 1: 4-8.

[26] Bruls, A., Cajot, L.G. and Franssen J.M. Characterization of an Insulating Material With Regard to ECCS Recommendations for the Fire Safety of Steel Structures. Journal of Constructional Steel Research 1988; 9: 111-135.

[27] BS5950, Structural Use of Steelworks in Building: Part 8, Code of Practice for Fire Resistance Design. British Standards Institution, London, UK, 1990.

[28] Burgess, I.W., Olawale, A.O. and Plank, R.J. Failure of Steel Columns in Fire. Fire Safety Journal 1992; 18: 183-201.

[29] Buress, I.W. and Najjar, S.R. A Simple Approach to the Behaviour of Steel Columns in Fire. Journal of Constructional Steel Research 1994; 31: 115-134.

[30] Burgess, R.A. Coatings for Fire Protection. Modern Steel Construction, December 1998: 64-70.

[31] Butcher, E.G. and Chitty, T.B. The Temperature Attained by Steel in Building Fires. JFRO, Fire Research Technical Paper No.15, London, HMSO, 1966.

[32] Butcher, E.G., Bedford, G.K. and Fardell, P.G. Further Experiments on Temperatures Reached by Steel in Building Fires. Proceedings of the Symposium held at the Fire Research Station, Boreham Wood, January, 1967: 45-56.

[33] Cajot L.G., Schleich, J.B. Global Fire Safety Concept for Buildings. International Conference on Composite Construction – Conventional & Innovative, Innsbruck, Australia, Sep. 16-18, 1994, pp 415-20.

[34] Cajot, L.G., Franssen , J.M. and Schleich, J.B. Computer Model for the Fire Resistance of Composite Structures. IABSE Symposium: Mixed Structures, including New Materials, Brussels 1990: 395-400.

[35] CEC, Construction Products Directive. Commission of European Community, N°L40/12, Luxembourg, December 1988.

[36] CEC, Interpretative Document No 2 /Essential Requirement on Safety in case of Fire. Commission of European Community, October 1993.

[37] CIB W14, A Conceptual Approach towards a Probability Based Design Guide for Structural Fire Safety. Fire Safety Journal 1983; 6 (1).

[38] CIB W14, Design-Guide – Structural Fire Safety. Fire Safety Journal 1986; 9: 77-136.

[39] Collier, P. Fire in a Residential Building: Comparisons between Experimental and Fire Zone Model. Fire Technology 1996; 32(3).

[40] Collier, P. Fire Modelling of Large Spaces. http://www.branz.org.nz/Branz/PandS/fireResearch/largespaces.htm, September 1998.

[41] Cooke, G.M.E., Lawson, R.M. and Newman, G.M. Fire Resistance of Composite Deck Slabs. The Structural Engineer 1988; 66 (16).

[42] Corotis, R.B. Load Combinations for Building Exposed to Fires. Proc. of National Steel Construction Conference, American Institute of Steel Construction, Missouri, March 14-17 1990: 8.1-8.19.

[43] Corradi, L., Poggi, C. and Setti, P. Interaction Domains for Steel Beam-Columns in Fire Conditions. Journal of Constructional Steel Research 1990; 17: 217-235.

[44] Culver, C.G. Survey Results for Fire Loads in Office Buildings. NBS, Building Science Series 85, National Burea of Standards, Washington D.C., 1976.

[45] Dhima, D., Fouguet, G., etc., Experimental and Numerical Assessment of the Behaviour of External Heavy Steel Columns Under Natural Fire. Journal of Constructional Steel Research 1998; 46 (1-3).

[46] Drysdale, D. An Introduction to Fire Dynamics. John Wiley & Sons Ltd., 1987.

[47] ECCS, Fire Safety in Constructional Steelwork. European Convention for Constructional Steelwork, CECM-III-74-2E, 1974.

[48] ECCS TC3, Fire Safety of Steel Structures. European Recommendations for the Fire Safety of Steel Structures, Elsevier, 1983.

[49] ECCS TC3, Fire Safety in Open Car Parks. European Convention for Constructional Steelwork, Technical Note No.75, Brussels, Belgium, 1993.

[50] Edwards, M. The Performance in Fire of Concrete Filled SHS Columns Protected by Intumescent Paint. Proceedings of International Conference on Nordic Steel Construction, Bergen, Norway, 1998: 27-40.

[51] El-Houssieny, O.M., Salam, S.A., Attia, G.A.M. and Saad, A.M. Behaviour of Extended End Plate Connections at High Temperature. Journal of Constructional Steel Research 1998; 46: 1-3.

[52] El-Rimawi, J.A., Burgess, I.W. and Plank, R.J. The Influence of Connection Stiffness on the Behaviour of Steel Beams in Fire. Journal of Constructional Steel Research 1997; 43(1-3): 1-15.

[53] El-Rimawi J.A., Burgess, I.W. and Plank, R.J. Studies of the Behaviour of Steel Subframes with Semi-Rigid Connections in Fire. Journal of Constructional Steel Research 1999; 49: 83-98.

[54] Emmons, H.W. Prediction of Fires in Building. 17th International Symposium on Combustion, the Combustion Institute, Pittsburgh, 1978.

[55] ENV 1991-2-2, Eurocode 1, Basis of Design and Actions on Structures, Part 2.2, Actions on Structures Exposed to Fire. Final edited version/CET/TC250/SC1, Brussels, November 1994.

[56] ENV 1993-1-2, Eurocode 3, Design of Steel Structures, Part 1.2: General Rules – Structural Fire Design. CEN/TC250/SC3, Brussels, September 1995.

[57] ENV 1994-1-1, Eurocode 4, Design of Composite Steel and Concrete Structures, Part 1.1: General Rules and Rules for Buildings. Final edited version/CEN/TC250/SC4, Brussels, November 1993.

[58] ENV 1994-1-2, Eurocode 4, Design of Composite Steel and Concrete Structures, Part 1.2: Structural Fire Design. Final edited version/CEN/TC250/SC4, Brussels, November 1993.

[59] Favre, J.P., Fontana, M. and Hass, R. Optimal Fire Safety through Modern Engineering Approaches. Seminar held in Helsinki University of Technolgy, Espoo, Finland,1995.

[60] FIP, FIP/CEB Report on Methods of Assessment of the Fire Resistance of Concrete Structural Members. FIP Commission on the Fire Resistance of Prestressed Concrete Structures, 1978.

[61] Fire Tests of the 140 William Street Office Building/The Effect of Fire in the Building at 140 William Street.

[62] Fitzgerald, R.W. and Lie, T.T. Structural Fire Protection. ASCE Manuals & Reports on Engineering Pratice, No.78, 1992.

[63] Fitzgerald, R.W. Building Fire Safety Engineering Method, Workbook, Worcester Polytechnic Institute, Worcester, Mass, 1993.

[64] Franssen, J.M., Cajot, L.G. and Schweppe, H. Connection between Parametric Temperature-Time Curves and Equivalent Time of Fire Exposure (Annex B and E of ENV-1991-2-2). IABSE Colloquium, Basis of Design and Actions on Structures, Delft, 1996: 407-417.

[65] Franssen, J.M., Cooke, G.M.E. and Latham, D.J. Numerical Simulation of Full Scale Fire Test on a Loaded Steel Framework. Journal of Constructional Steel Research 1995; 35: 377-408.

[66] Franssen, J.M., Talamona, D., Kruppa, J. and Cajot, L.G. Stability of Steel Columns in case of Fire: Experimental Evaluation. Journal of Structural Engineering 1998; 124(2): 158-163.

[67] Franssen, J.M. and Dotreppe, J.C. Fire Resistance of Columns in Steel Frames. Fire Safety Journal 1992; 19: 159-175.

[68] Franssen, J.M., Schleich, J.B., Cajot, L.G. and Azpiazu, W. A Simple Model for the Fire Resistance of Axially Loaded Members - Comparison with Experimental Results. Journal of Constructional Steel Research 1996; 37 (3): 175-204.

[69] Hagen, E. and Haksever, A. A Contribution for the Investigation of Natural Fires in Large Compartment. Fire Safety Science- Proceedings of the First International Symposium, Berkeley, Califonia, November 1985: 149-158.

[70] Haksever, A. Fire Engineering Design of Steel Framed Car-park Buildings. Department of Structural Engineering, Helsinki University of Technology, Report No.112, Espoo, Finland, 1990.

[71] Hamerlinck, R. and Twilt, L., Fire Resistance of Composite Slabs. Journal of Constructional Steel Research, Special Issue on Fire and Steel 1995; 33 (1-2): 71-86.

[72] Harda, K. and Terai, T. Dependence of Thermal Response of Composite Slabs Subjected to Fire on Cross Sectional Shapes. Fire Safety Science – Proceedings of the Fourth International Symposium, International Association for Fire Safety Science, 1995: 1159-1170.

[73] Harmathy, T.Z. A New Look at Compartment Fire. Fire Technology 1972; 8 (3): 197-217; 8 (4): 327-351.

[74] Harmathy, T.Z. Fire Severity: Basis of Fire Safety Design. Presented at the International Simposium of Fire Safety of Concrete Structures, Fall Convention of the American Concrete Institute, San Juan, September 21-26, 1980.

[75] Harmathy, T.Z. and Mehaffey, J.R. Normalized Heat Load: A Key Parameter in Fire Safety Design. Fire and Materials 1982; 6 (1): 27-31.

[76] Heselden, A.J.M. Parameters Determining the Severity of Fire. Proceedings of the Symposium held at the Fire Research Station, Boreham Wood, January, 1967: 19-28.

[77] Ingberg, S.H. Test of Severity of Building Fires. Quarterly, National Fire Association, vol.22, 1928.

[78] ISO, Fire Resistance Tests - Elements of Building Construction. International Organisation for Standardisation, ISO-834 Part 1, Switzerland, 1992.

[79] Johnson, P.F. International Developments in Fire Engineering of Steel Structures. Journal of Constructional Steel Research 1998; 46 (1-3).

[80] Jones, W.W., Forney, G.P., Baily, J.L. and Tatem, P.A. Comparison of CFAST Prediction to Real-Scale-Fire Tests. National Institute of Standards and Technology, Naval Technology Center for Safety and Surivivability, 1996.

[81] Kawagoe, K. Fire Behavior in Rooms. Building research Institute, Japan, Report No.27, 1958.

[82] Kawagoe, K. and Sekine, T. Estimation for Fire Temperature-time Curve in Rooms. BRI Occasional Report No.11, Building Research Institute, Japan, 1963.

[83] Kemp, K.O. Yield of a Square Reinforced Concrete Slab on Simple Supports, Allowing for Membrane Forces. The Structural Engineer 1967; 45 (7): 235-240.

[84] Kirby, B.R. Recent Developments and Applications in Structural Fire Engineering Design- A Review. Fire Safety Journal 1986; 11: 141-179.

[85] Kirby, B.R. British Steel Technical European Fire Test Programme – Design, Construction and Results. Proceedings of the 2nd Cardington Conference, 1996: 111-126.

[86] Kordina, K. Natural Fires: Research and Application in Consulting. Fire Safety Journal 1991; 17: 115-130.

[87] Kouhi, J. Euroopan Hiili-ja Tersunionin (ECSC) T&K-toiminta. Presented in Teräsrakenteiden Tutkimus- Ja Kehityspäivät, August 25-26, 1998.

[88] Kruppa, J. and Zhao, B. Fire Resistance of Composite Beams to Eurocode 4 part 1.2. Journal of Constructional Steel Research, Special Issue on Fire and Steel 1995; 33 (1-2): 51-70.

[89] Law, M. A Basis for the Design of Fire Protection of Building Structures. Structural Engineer, 1983; 61A(1):25-33.

[90] Law, M. and Arnault, P. Fire Loads, Natural Fires and Standard Fires. International Conference on Planning and Design of Tall Buildings Preprints: Reports vol. 1-8, 1972, ASCE-IABSE.

[91] Law, M. Prediction of Fire Resistance. Paper No.2 of Symposium No.5: Fire Resistance Requirements for Buildings – A New Approach, London, HMSO, 1973

[92] Law, M. Design for Fire Safety of Bare Exterior Structural Steel: 1. Theory and Validation. 2. State of the Art, Ove Arup and Partners, London, 1977.

[93] Law, M. A Basis for the Design of Fire Protection of Building Structures. The Structural Engineer 1983; 61A (1): 25-33.

[94] Law, M. Fire Grading and Fire Behaviour. Fire Safety Journal 1991; 17 (2): 147-153.

[95] Lawson, R.M., Mullett, D.L. and Rackham, J.W. Design of Asymmetric Slimflor Beams using Deep Composite Decking. The Steel Construction Institute, Publication P175, 1997.

[96] Lawson, R.M., and Blaffart, H. Fire Safe Design of Composite Deck Slabs. Structural Engineering International 1996; 2: 119-122.

[97] Lawson, R.M. Fire Resistance of Slim Floor Construction. Proceedings of International Conference on Steel and Composite Structures, Delft, The Netrherland 1999: 9.1-9.14.

[98] Lennon, T. Large Compartment Fire Tests. Fire, Static and Dynamic Tests of Building Structures, Proceedings of the Second Cardington Conference, England, March 1996: 45-53.

[99] Li, G.Q., Jiang, S.C., Lin, G.X. and Yu, L.H. An Experimental Research on Behaviour of Steel Frames Subjected to Fires. Proceedings of the 3rd Sino-Japan Conference on Structural Technology, Shen Zhen, P.R.China, November 4-8, 1997.

[100] Lie, T.T. and Chabot, M. A Method to Predict the Fire Resistance of Concrete Filled Hollow Steel Columns. Journal of Fire Protection Engineering 1990; 2 (4); 111-126.

[101] Lie, T.T. and Irwin, R.J. Fire Resistance of Rectangular Steel Columns Filled with Bar-Reinforced Concrete. Journal of Structural Engineering 1995; 121(5): 797-805.

[102] Liew, J.Y.R, Tang, L.K., Holmas, T. and Choo, Y.S. Advanced Analysis for the Assessment of Steel Frames in Fire. Journal of Constructional Steel Research 1998; 47 (1-2): 19-45

[103] Liu, T.C.H. Effect of Connection Flexibility on the Fire Resistance of Steel Beams. Journal of Constructional Steel Research 1998; 45 (1): 99-118.

[104] Liu, T.C.H. Finite Element Modelling of Behaviours of Steel Beams and Connections in Fire. Journal of Constructional Steel Research 1996; 36 (3): 181-199.

[105] Lu, X. and Mäkeläinen, P. Strength and Stiffness of Composite Slim Floor Beams. Structural Steel, PSSC'95, 4th Pacific Structural Steel Conference, Volume 3, Steel-Concrete Composite Structures, Singapore, 1995: 93-100.

[106] Lu, X. Study on the Structural Behaviour and Composite Action in Composite Slim Floor Beams. Acta Polytechnica Scandinavica, Civil Engineering and Building Construction Series No. 103, Helsinki, 1995.

[107] Lu, X. and Mäkeläinen, P. Slim Floor Development in Sweden and Finland. Structural Engineering International 1996; 2: 127-129.

[108] Loikkanen, P. and Mangs, J. Characterization of the Fire Behaviour of a Burning Passenger Car. Fire Technology Laboratory VTT, Reports PAL 00455/90 and PAL 00455A/90, Finland, 1992.

[109] Ma, Z. Damage Evaluation and Aseismic Repair of R.C. Structures After Fire. Research Report No.3, Steel Structures in Helsinki University of Technology, Espoo, Finland, 1998.

[110] Ma, Z. and Mäkeläinen, P. Temperature Analysis of Steel-Concrete Composite Slim Floor Structures Exposed to Fires. Laboratory of Steel Structures Publications 10, Helsinki University of Technology, Espoo, 1999.

[111] Ma, Z. and Mäkeläinen, P. Numerical Analysis of Steel-Concrete Composite Slim Floor Structures in Fire. Laboratory of Steel Structures Publications 11, Helsinki University of Technology, Espoo, 1999.

[112] Ma, Z. and Mäkeläinen, P. Fire Resistance of Hybrid Steel Beam in Composite Slim Floor Structures Construction. Teräsrakenteiden Tutkimus- ja Kehityspäivät, Teknillinen Korkeakoulu, Tammikuu 2000, pp.53-60.

[113] Ma, Z. and Mäkeläinen, P. Fire Safety Design of Composite Floor System. Teräsrakenteiden Tutkimus- ja Kehityspäivät, Lappeerannan Teknillinen Korkeakoulu, August 1998, Finland.

[114] Ma, Z. Fire Engineering Design of Composite Floor System. Seminar on Steel Structures: Design of Steel-Concrete Composite Structures. Laboratory of Steel Structures Helsinki University of Technology, TKK-TER-4, 1998.

[115] Ma, Z. Modern Fire Safety Design Methods. p.78 (unpublished).

[116] Ma, Z. Structural Behaviour of Composite Slim Floor Frame in Fire Conditions. P.23 (unpublished).

[117] Ma, Z., Wu, B. and Ou, J. Fully-Developed Compartment Fire Model and Its Application. Journal of Harbin University of Civil Engineering and Architecture 1998; 31 (1): 16-24.

[118] Ma, Z., Wu, B. and Ou, J. Literature Review of Fire Engineering Design of R.C. Structures. Building Structures (Chinese Journal) 1997; 11: 48-52.

[119] Ma, Z., Wu, B. and Ou, J. Seismic Characteristics of R.C. Columns after Fire. Chinese Journal of Computational Mechanics 1997; 14 (4): 443-453.

[120] Ma, Z. and Wu, B. Evaluation Technology of Fire- Damaged R.C. Structures. Institute of Engineering Theory and Its Application, Harbin University of Civil Engineering and Architecture, Research Report, 1997, 126p.

[121] Magns, J. and Keski-Rahkonen, O. Characterization of the Fire Behaviour of a Burning Passenger Car. Part I: Car Fire Experiments. Fire Safety Journal 1994; 23: 17-35.

[122] Magns, J. and Keski-Rahkonen, O. Characterization of the Fire Behaviour of a Burning Passenger Car. Part II: Parametrization of Measured Rate of Heat Release Curves. Fire Safety Journal 1994; 23: 37-49.

[123] Magnusson, S.E. and Thelandersson, S. Temperature-time Curve for the Complete Process of Fire Development, A Theoretical Study on Wood Fuel Fires in Enclosed Space. Acta Polytechnic Scandinavia, Civil Engineering and Building Construction Series No.65, 1970.

[124] Magnusson, S.E. and Thelandersson, S. Comments on Rate of Gas Flow and Rate of Burning for Fires in Enclosures, Division of Structural Mechanics and Concrete Construction, Lund Institute of Technology, Sweden, Bulletin 19, 1971.

[125] Malhotra, H.L. Design of Fire-Resisting Structures. Surrey University Press, 1982.

[126] Mehaffey, J.R. and Harmathy, T.Z. Thermal Response of Compartment Boundaries to Fire. Fire Safety Science-Proceedings of the First International Symposium, Berkley, California, November 1985.

[127] McCaffery, B.J., Quintiere, J.G. and Harklerroad. Estimating Room Temperatures and the Likelihood of Flashover Using Test Data Correlation. Fire Technology 1981; 17 (2): 98-119.

[128] Mullett, D.L., Lawson, R.M. Slim Floor Construction Using Deep Decking. SCI P127, The Steel Construction Institute, Berkshire, United Kingdom, 1993.

[129] Myllymäki, J. and Kokkala, M. Fire Tests of a Steel Beam Under Ceiling Exposed to a Localized Fire. Fifth Finnish Steel Structures R&D Days, 18-19 January 2000: 146-153.

[130] Neves, I.C. The Critical Temperature of Steel Columns with Restrained Thermal Elongation. Fire Safety Journal 1995; 24: 211-227.

[131] Newman, G.M. Fire Resistance of Slim Floor Beams. Journal of Constructional Steel Research 1995; 33 (1-2): 87-100.

[132] Newman, G.M. Design Implications of the Cardington Fire Research Programme. Proceedings of the Second Cardington Conference, Cardington, England, 1996; 161-168.

[133] Newman, G.M. Fire Resistance of Slim Floor Beams. Journal of Constructional Steel Research 1995; 33 (1-2): 87-100.

[134] Newman, G.M. and Walker, H.B., Design Recommendation for Composite Floors and Beams Section 2: Fire Resistance. Steel Construction Institute, 1983.

[135] NFPA/SFPE, Handbook of Fire Protection Engineering (2nd Edition). Society of Fire Protection Engineers, National Fire Protection Association, 1995.

[136] Nilsson, L. Time Curve of Heat Release for Compartment Fires with Fuel of Wooden Cribs. Division of Structural Mechanics and Concrete Construction, Lund Institute of Technology, Sweden, Bulletin 36, 1974

[137] Nullifire S607 – Fire Painting, Certified Directions for Use (in Finnish), TRY-18-1996, Helsinki, 1996.

[138] Oksanen, T. and Ala-Outinen, T. Determination of Fire Resistancs of Concrete Filled Hollow Steel Sections. Technical Research Center of Finland , Research Notes 1287, Espoo, 1991.

[139] Ove Arup & Partners, Design Guide for Fire Safety of Bare Exterior Structural Steel. Technical Reports, American Iron and Steel Institute, Constrado, London, 1977.

[140] O'Connor, M.A. Numerical Modelling of Composite Structures Subjected to Thermal Loading. Proc. of the Second Cardington Conference, Cardington, England, 1996: 127-131.

[141] Paulsen, O.R. and Hadvig, S. Heat Transfer in Fire Test Furnaces. Journal of Fire and Flammability, vol.8, 423-442

[142] Pettersson, O. The Possibilities of Predicting the Fire Behaviour of Structures on the Basis of Data from Standard Fire-Resistance Tests. Lund Institute of Technology, Lund, Sweden, 1971, 43p.

[143] Pettersson, O., Magnusson, S.E. and Thor, J. Fire Engineering Design of Steel Structures. Swedish Institute of Steel Construction, Publication 50, Stockholm, Sweden, 1976.

[144] Pettersson, O. Rational Structural Fire Engineering Design, Based on Simulated Real Fire Exposure. Fire Safety Science, Proceedings of the Fourth International Symposium , June 1994: 1-24.

[145] Plank, R., Burgess, I. and Bailey, C. Modelling the Behaviour of Steel-framed Building Structures by Computer. Proceedings of the 2nd Cardington Conference, 1996: 145-159.

[146] Poh, K.W. and Bennets, I.D. Behaviour of Steel Columns at Elevated Temperatures. Journal of Structural Engineering 1995; 121(4): 676-684.

[147] Poh, K.W. and Bennets, I.D. Analysis of Structural Members under Elevated Temperature Conditions. Journal of Structural Engineering 1995; 121(4): 664-675.

[148] Purkiss, J.A. Developments in the Fire Safety Design of Structural Steelwork. Journal of Constructional Steel Research 1988; 11: 149-173

[149] Ramachandran, G. Economic Value of Automatic Fire Dectors. BRE Information Paper 1. pages 27-80, Garston, 1980.

[150] Robinson, J. Fire - A Techincal Challenge and a Market Opportunity. Journal of Constructional Steel Research 1998; 46 (1-3): Paper No.179.

[151] Robinson, J. and Newman, G.M. Cardington Fire Tests: First Results. New Steel Construction 1997; 5 (3): 23-27.

[152] Rocket, J.A. Modelling of NBS Test with the Harvard Mark V Fire Simulation. NBSIR, 81-2240, 1980.

[153] Rose, P.S., Bailey, C.G., Burgess, I.W. and Plank, R.J. The Influence of Floor Slabs on the Structural Performance of the Cardington Frame in Fire. Journal of Constructional Steel Research 1998; 46 (1-3): Paper No.181.

[154] Schleich, J.B. and Scherer, M. Compartment Temperature Curves in Function of Opening Factor and Fire Load. ECSC research 7210-SA/112, Activity C1/ RPS Report No 08/90, Luxembourg, 1991.

[155] Schleich, J.B. and Cajot, L.G. Natural Fire in Large Compartments. ECSC Research 7210-SA/517etc., B_E_F_L_NL, 1993-96.

[156] Schleich J.B. Competive Steel Buildings through Natural Fire Safety Concept. ECSC Research 7210-SA/522 betc, B_D_E_F_I_L_NL_UK &ECCS, 1994-98.

[157] Schleich, J.B. A Natural Fire Safety Concept For Buildings –1. Fire, Static and Dynamic Tests of Building Structures, Proceedings of the 2nd Cardington Conference, Cardington, England, 12-14 March 1996: 79-104.

[158] Schleich, J.B. Eurocode 1/part2.2: Actions on Structures Exposed to Fire. IABSE Colloquium, Basis of Design and Actions on Structures, Delft, 1996: 71-81.

[159] Schleich, J.B. Global Fire Safety Concept According to the Eurocodes. Proceedings of Nordic Steel Construction Conference '95, volume II, Malmö, Sweden, June 19-21, 1995: 667-76.

[160] Schleich, J-B. Global Fire Safety Concept for Buildings: Connection to ECSC Research, ECCS Developments and CEN Standardization. In: Proceedings of International Conference on Steel and Composite Structures, Delft, The Netherlands, 1999:8.1-8.26.

[161] Schleich, J.B. Fire Safety, Design of Composite Columns. Presented in the international conference on Fire Safe Steel Construction: Practical Design, Luxembourg, April 1984.

[162] Schleich, J.B. Computer Assisted Analysis of the Fire Resistance of Steel and Composite Concrete-Steel Structures. REFAO-CAFIR, Luxembourg, 1987, 293p.

[163] SCI, Structural Fire Engineering: Investigation of Broadgate Phase 8 Fire. Steel Construction Institute, UK, 1991.

[164] Sha, W. Fire Resistance of Floors Constructed with Fire-Resistant Steels. Journal of Structural Engineering 1998;124 (6): 664-670.

[165] Tanaka, T., Sato M. and Wakamatsu, T. Simple Formula for Ventilation Controlled Fire Temperatures. Fire Science and Technology, Special Issue for Kawagoe Memorial Symposium 1997; 17: 15-27.

[166] Tanaka,T. presented in the Proceedings of Janpanese Institute of Fire, 1979; 29 (2), 1980; 30 (1, 2).

[167] Tanaka, T. A Model of Multiroom Fire Spread. Fire Science and Technology 1983; 3 (2): 105-121.

[168] Thomas, P.H. and Webster, C.T. Some Experiments on Buoyant Diffusion Flames. Combustion and Flame 1961; 5: 359p.

[169] Thomas, P.H. Fire in Model Rooms: CIB Research Programmes, BRE current paper CP32/74, Borehamwood, 1974.

[170] Thomas, P.H. Some Problems Aspects of Fully Developed Room Fires. Fire Standards and Safety, ASTM, SP-614, Aprile 5-6, 1976, pages 112-130.

[171] Thomas, P.H. and Heselden, A.J.M. Fully-developed Compartment Fires, Two Kinds of Behaviour. Fire Research Technology, Paper No.18, Joint Fire Research Organization, England, 1967.

[172] Thomas, P.H. and Law, M. Fire Prevention Science and Technology 1974; 10: 19-26.

[173] Thomas, P.H. and Heselden, A.J.M. Fully Developed Fire in Single Compartments. A Cooperative Research Programme of the Conseil Internationale du Batiment. CIB Report No.20, Fire Research Note No.923, 1972.

[174] Thomson, G. and Preston, R.R. Towards Harmonized Standard Fire Resistance Testing. Fire Safety Journal 1996; 27: 91-112.

[175] Translated by Sun, J.X., etc. Global Fire Safety Design for Building Structures. By Japanese Constructional Ministry (1983). Scie. & Techn. Translation Ltd in Tian Jin, China, 1994.

[176] Tschemmernegg, F. Composite Frames with Slim Floors. Journal of Constructional Steel Research 1998; 46 (1-3).

[177] Twilt, L. and Witteveen, J. Calculation Methods for Fire Engineering Design of Steel and Composite Structures. Proceedings of International Conference on Design of Structures Against Fire, held at Aston University, Birminham, 1986: 155-176.

[178] Twilt, L., Cajot, L.G. etc. Input Data for the Natural Fire Design of Building Structures. IABSE Colloquium, Basis of Design and Actions on Structures, Deft 1996: 395-405.

[179] Underwriters Laboratory Inc. Fire Severity at the Exterior of a Burning Building. Technical Report, American Iron and Steel Institute, Washington, D.C., April 1975.

[180] Wang, Y., Cooke, G. and Moore, D. Large Compartment Fire Tests at Cardington and the Assessment of Eurocode 1. IABSE Colloquium, Basis of Design and Actions on Structures, Delft 1996: 419-427.

[181] Wang, Y.C. Some Considerations in the Design of Unprotected Concrete-Filled Steel Tubular Columns Under Fire Conditions. Journal of Structural Steel Research 1997; 44 (3): 203-223.

[182] Wang, Y.C. Tensile Membrane Action in Slabs and Its Application to the Cardington Fire Tests. Proceedings of the 2nd Cardington Conference, 1996: 55-67.

[183] Weatherford, W.D. Scaling of Flames Above Free-Burning Structural Models. Combustion and Flame, Vol.14, 1970, 21p.

[184] Webster, C.T. and Smith, P.G. The Burning of Well Ventilated Compartment Fires, Part IV, Brick Compartment, 2.4 m Cube. JFRO, Fire research Note No.578, 1964

[185] Witteveen, J. and Twilt, L. A Critical View on the Results of Standard Fire Resistance Tests on Steel Columns. Fire Safety Journal 1981/82; 4.

[186] Williamson, R.B. and Dembsey, N.A. Advances in Assessment Methods for Fire Safety. Fire Safety Journal 1993; 20: 15-38.

[187] Wong, M.B. and Patterson, N. Unit Load Factor Method for Limiting Temperature Analysis of Steel Frames with Elastic Buckling Failure Mode. Fire Safety Journal 1996; 27: 113-122.

[188] Wu, B., Ma, Z. and Ou, J. Experimental Study on Seismic Properties of R.C. Columns after High Temperature. China Civil Engineering Journal 1999; 32 (2): 53-58.

[189] Wu, B., Ma, Z. and Ou, J. Seismic Behavior of Fire-Damaged R.C. Columns. Earthquake Engineering and Engineering Vibration (Chinese Journal, English version), Special Issue for 30th World Conference of Geology 1996; 16: 76-86.

[190] Wu, B., Ma, Z. and Ou, J. Stress-Strain Curves of Fire-Damaged Concrete under Cyclic Axial Loading. Earthquake Engineering and Engineering Vibration (in Chinese) 1997; 17 (3).

[191] Wu, B., Ma, Z. and Ou, J. Experimental Study on Deformation and Constitutive Relationship of Concrete under Axial Loading and High Temperature. Chinese Journal of Building Structures 1999; 20 (5); 42-49.

[192] Wu, B., Ma, Z. and Ou, J. A Study on Seismic Behavior of Fire-Damaged R.C. Compression-Flexural Components. Earthquake Engineering and Engineering Vibration (Chinese Journal) 1994; 14 (4): 24-34.

[193] Wu, B., Li, H. and Ma, Z. Damage Analysis of Fire –Damaged R.C. Structures under Low-Cyclic Loading. Journal of Harbin University of Civil Engineering and Architecture 1997; 30 (3): 7-12.

[194] Wu, B., Ma, Z., etc. Fire Disaster and Its Retrofit of Shenyang Department Store. Chinese Journal of Catastrophe 1996; 11 (4): 48-53.

[195] Wu B. and Ma, Z. Damage Assessment and Seismic Retrofit of R.C. Structures after Fire. Theories and Practices of Structural Engineering, Seismological Press - Proceedings of the 5th International Symposium on Structural Engineering for Young Experts, Shenyang, P.R. of China, 1998: 599-607.

[196] Yokoi, S. Study of the Prevention of Fire Spread Caused by Hot Upward Currents. Building Research Institute, Report No.34, Tokyo.

[197] Ödeen, K. Theoretical Study of Fire Characteristics in Enclosed Spaces. Royal Institute of Technology Division of Building Construction, Bulletin, No.11, 1963.

[198] Ödeen, K. Temperature Development in Externally Fire Exposed Rooms. Fire Safety Journal 1991; 17: 165-170.

Index of the Author's Publications

1. Articles in scientific journals

1. **Ma, Z.** and Mäkeläinen, P., Behaviour of Composite Slim Floor Structures in Fire. Journal of Structural Engineering, ASCE, Vol. 126, No.7, pp.830-837, July 2000.
2. **Ma, Z.** and Mäkeläinen, P., Parametric Temperature-time Curves of Medium Compartment Fires for Structural Design. Journal of Fire Safety, Vol. 34, No. 4, pp.361-375, June 2000.
3. Mäkeläinen, P. and **Ma, Z.**, Fire Resistance of Composite Slim Floor Beams. Journal of Constructional Steel Research, Vol. 54, No. 3, pp.345-363, June 2000.
4. Wu, B., **Ma, Z.** and Ou, J., Experimental Study on Seismic Properties of R.C. Columns after High Temperature. China Civil Engineering Journal, Vol.32, No.2, 1999, pp53-58.
5. **Ma, Z.**, Wu, B. and Ou, J., Seismic Characteristics of R.C. Columns after Fire. Chinese Journal of Computational Mechanics, Vol.14, No.4, 1997, pp443-453.
6. Wu, B., **Ma, Z.** and Ou, J., Seismic Behavior of Fire-Damaged R.C. Columns. Earthquake Engineering and Engineering Vibration (Chinese Journal, English version), Special Issue for 30th World Conference of Geology, Vol.16, 1996, pp76-86.
7. Wu, B., **Ma, Z.** and Ou, J., Stress-Strain Curves of Fire-Damaged Concrete under Cyclic Axial Loading. Earthquake Engineering and Engineering Vibration (in Chinese), Vol.17, No.3, 1997.
8. Wu, B., **Ma, Z.** and Ou, J., Stress-Strain Curves of Fire-Damaged Concrete Under Static Axial Loading. Chinese Journal of Building Structures 1999; 20 (5); 42-49.
9. Wu, B., **Ma, Z.** and Ou, J., A Study on Seismic Behavior of Fire-Damaged R.C. Compression-Flexural Components. Earthquake Engineering and Engineering Vibration (Chinese Journal), Vol.14, No.4, 1994, pp24-34
10. **Ma, Z.**, Wu, B. and Ou, J., Fully-Developed Compartment Fire Model and Its Application. Journal of Harbin University of Civil Engineering and Architecture, Vol.31, No.1, 1998, pp16-24
11. Wu, B., Li, H. and **Ma, Z.**, Damage Analysis of Fire –Damaged R.C. Structures under Low-Cyclic Loading. Journal of Harbin University of Civil Engineering and Architecture, Vol.30, No.3, 1997, p7-12
12. **Ma, Z.**, Wu, B. and Ou, J., Literature Review of Fire Engineering Design of R.C. Structures. Building Structures (Chinese Journal), No.11, 1997, pp48-52

13. Wu, B., **Ma, Z.**, etc., Fire Disaster and Its Retrofit of Shenyang Department Store. Chinese Journal of Catastrophe, Vol.11, No.4, 1996, pp48-53

14. **Ma, Z.**, Wang, L. and Wu, Z., Aseismic Behavior of Composite Steel-Concrete Frame – State of the Art. Journal of Harbin University of Civil Engineering and Architecture, Vol.29, No.6, 1995, pp132-138

15. **Ma, Z.** and Wu, Z., Elastic Stiffness Matrix of Fully Connected Steel-Concrete Composite Beams. Journal of Harbin University of Civil Engineering and Architecture, Vol.28, No.5, 1995, pp235-241

16. **Ma, Z.** and Wu, Z., A Mechanical Model on the Stud Connectors in Composite Beams. Journal of Harbin University of Civil Engineering and Architecture, Vol.28, No.5, 1995, pp242-248

17. **Ma, Z.** and Wu, Z., A Parametric Study on the Effective Width of Steel-Concrete Composite Beams – A New View. Journal of Harbin University of Civil Engineering and Architecture, Vol.28, No.5, 1995, pp249-255

2. Articles in scientific conference proceedings

18. **Ma, Z.** and Mäkeläinen, P., Temperature Analysis of Steel-Concrete Composite Slim Floor Structures Exposed to Fire. The 6th. ASCCS International Conference on Steel-Concrete Composite Structures - The Past and the Coming Century, March 22-24, 2000, Los Angeles, Volume I, pp.263-270.

19. Wu B. and **Ma, Z.**, Damage Assessment and Seismic Retrofit of R.C. Structures after Fire. Theories and Practices of Structural Engineering, Seismological Press - Proceedings of the 5th International Symposium on Structural Engineering for Young Experts, Shenyang, P.R. China, 1998, pp599-607

20. **Ma, Z.** and Mäkeläinen, P., Fire Resistance of Hybrid Steel Beam in Composite Slim Floor Construction. Teräsrakenteiden Tutkimus- ja Kehityspäivät, Teknillinen Korkeakoulu, Tammikuu 2000.

21. **Ma, Z.** and Mäkeläinen, P., Fire Safety Design of Composite Floor System. Teräsrakenteiden Tutkimus- Ja Kehityspäivät, Lappeerannan teknillinen korkeakoulu, Aug. 1998, Finland.

3. University publications

22. Malaska M., **Ma, Z.** and Mäkeläinen P., Steel-Concrete Composite Slim Floor Frame System. TKK-TER-14, Laboratory of Steel Structures Publications 14, Helsinki University of Technology, Espoo, 2000, p53.

23. **Ma, Z.** and Mäkeläinen, P., Numerical Analysis of Composite Steel-Concrete Slim Floor Structures in Fire. TKK-TER-11, Laboratory of Steel Structures Publications 11, Helsinki University of Technology, Espoo, 1999, p47.

24. **Ma, Z.** and Mäkeläinen, P., Temperature Analysis of Steel-Concrete Composite Slim Floor Structures Exposed to Fire. TKK-TER-10, Laboratory of Steel Structures Publications 10, Helsinki University of Technology, Espoo, 1999, p47.

25. **Ma, Z.**, Fire Engineering Design of Composite Floor System. Seminar on Steel Structures: Design of Steel-Concrete Composite Structures, Mikko Malaska etal. Laboratory of Steel Structures, Helsinki University of Technology, Report 4, Espoo, 1998, pp88-104

26. **Ma, Z.**, Damage Evaluation and Aseismic Repair of R.C. Structures after Fire. TKK-TER-3, Laboratory of Steel Structures Publications 3, Helsinki University of Technology, Espoo, 1998, p75

27. **Ma, Z.** and Wu, B., Evaluation Technology of Fire- Damaged R.C. Structures. Institute of Engineering Theory and Its Application, Harbin University of Civil Engineering and Architecture, Research Report, 1997, p126

28. Ou, J., Duan, Z., **Ma, Z.**, et al. Studies on the Ice-Vibrated Controls of Offshore Structures Using Viscous-Elastic and Hydraulic Dampers – Numerical Analysis. Internal Research Report, Institute of Engineering Theory and Its Application, Harbin University of Civil Engineering and Architecture, July, 1997, p420.

29. Mäkeläinen, P. and **Ma, Z.**, Vertical Deflection of Buried Corrugated Steel Pipes. TeRT-99-05, Laboratory of Steel Structures, Helsinki University of Technology, Espoo, 1999.

4. Computer programs

Ma, Z., Temperature Analysis of Composite Structures Exposed to FIRe (**TACS-FIR**). Laboratory of Steel Structures, Helsinki University of Technology, Nov., 1998.

Ma, Z., Finite Element Analysis of Steel-Concrete Composite Beam - for some special problems (**FEA-SCCB**). Department of Structural Engineering, Harbin University of Civil Engineering and Architecture, 1994.

HELSINKI UNIVERSITY OF TECHNOLOGY LABORATORY OF STEEL STRUCTURES PUBLICATIONS

TKK-TER-4 Malaska, M., Korhonen, E., Vuolio, A., Lu, W., Outinen, J., Myllymäki, J., Ma, Z.,
Seminar on Steel Structures: Design of Steel-Concrete Composite Structures, 1998.

TKK-TER-5 Tenhunen, O.,
Ohutlevyorsien asennustarkkuus, 1998.

TKK-TER-6 Sun, Y.,
The Shear Behaviour of a Composite Floor Slab with Modified Steel Sheeting Profile, 1998.

TKK-TER-7 Lu, W., Kesti, J., Mäkeläinen, P.,
Shear and Cross-Tension Tests for Press-Joins, 1998.

TKK-TER-8 Lu, W., Segaro, P., Kesti, J., Mäkeläinen, P.,
Study on the Shear Strength of a Single-Lap Rosette-Joint, 1998.

TKK-TER-9 Malaska, M., Mäkeläinen, P.,
Study on Composite Slim Floor Beams, 1999.

TKK-TER-10 Ma, Z., Mäkeläinen, P.,
Temperature Analysis of Steel-Concrete Composite Slim Floor Structures Exposed to Fire, 1999

TKK-TER-11 Ma, Z., Mäkeläinen, P.,
Numerical Analysis of Steel-Concrete Composite Slim Floor Structures in Fire, 1999.

TKK-TER-12 Kaitila, O.,
The Behaviour of Steel-Concrete Composite Frames in Fire Conditions, 1999.

TKK-TER-13 Kesti, J., Mäkeläinen, P. (toim./ eds.),
Viidennet teräsrakenteiden tutkimus- ja kehityspäivät 18.-19.1.2000.

TKK-TER-14 Malaska, M., Ma, Z., Mäkeläinen, P.,
Study on Composite Slim Floor Beams, 2000.

TKK-TER-15 Hara, R., Kaitila, O., Kupari, K., Outinen, J., Perttola, H.
Seminar on Steel Structures: Design of Cold-Formed Steel Structures, 2000.

TKK-TER-16 Lu, W.
Neural Network Model for Distortional Buckling Behaviour of Cold-Formed Steel Compression Members, 2000.

TKK-TER-17 Kaitila, O., Kesti, J., Mäkeläinen, P.
Rosette-Joint and Steel Trusses, 2000.

ISBN 951-22-5222-8

ISSN 1456-4327